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# Engineering News-Record

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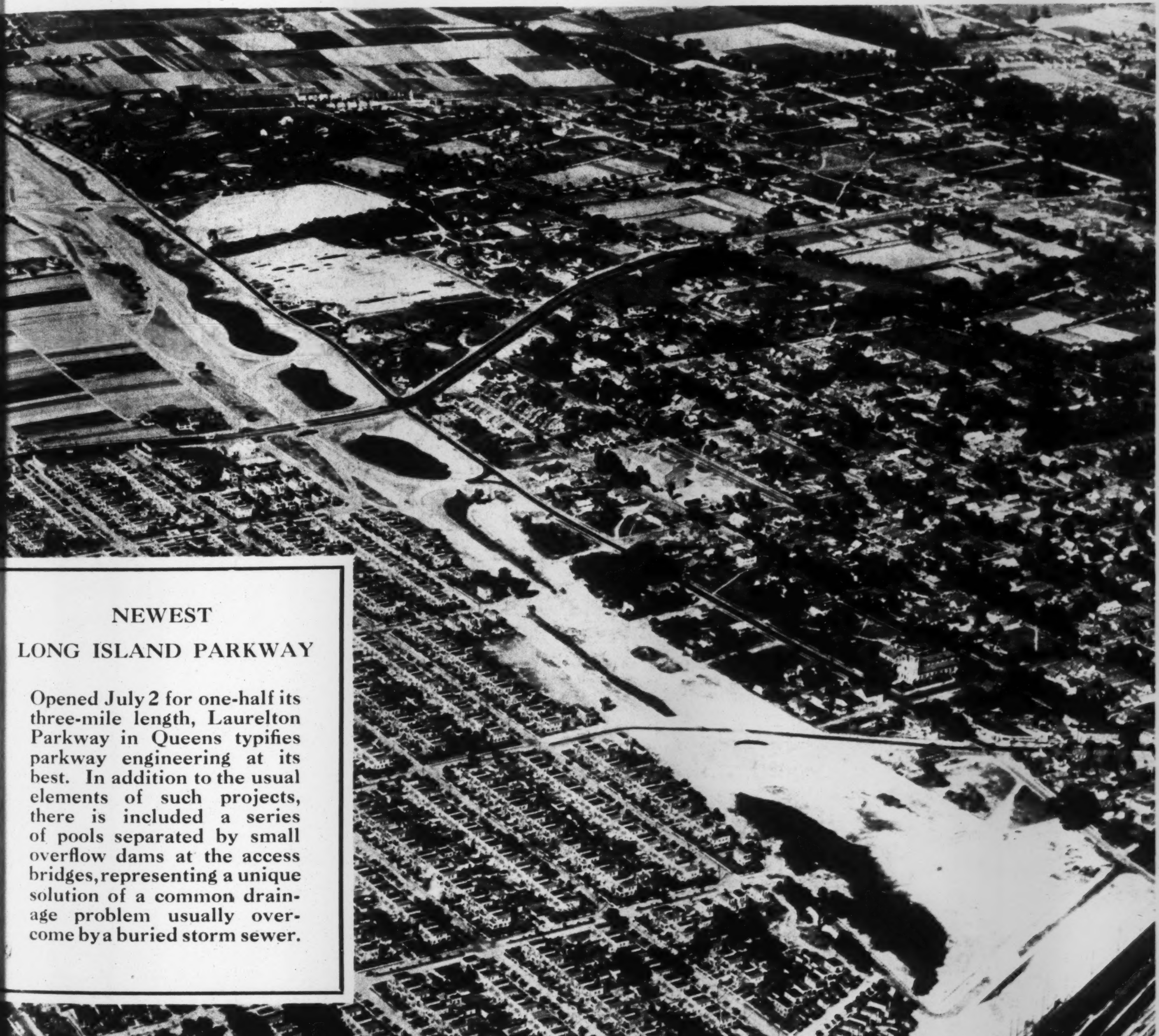
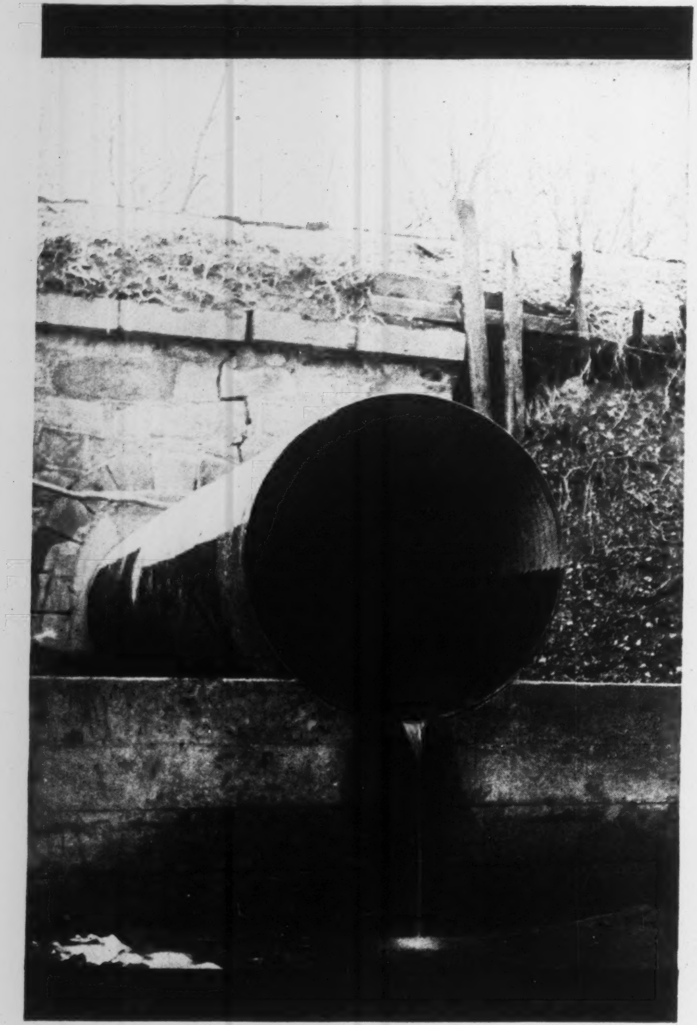


## NEWEST LONG ISLAND PARKWAY

Opened July 2 for one-half its three-mile length, Laurelton Parkway in Queens typifies parkway engineering at its best. In addition to the usual elements of such projects, there is included a series of pools separated by small overflow dams at the access bridges, representing a unique solution of a common drainage problem usually overcome by a buried storm sewer.

*Aerial Explorations Photo*

IN THIS ISSUE . . . Engineering Aspects of the New York Flood . . . New Bridges on Boston Shore Hi ways . . . Substructures for Great Lakes Lighthouses . . . Hydraulic-Fill Levees at Lake Okeechobee



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# To Foster Normal Re-employment

LAST week this page recorded progress toward putting the work-relief program on a more practical basis. It cannot be too strongly emphasized, however, that this progress is largely potential; if it is to become reality the industry still has a job ahead of it. Its constant vigilance and effort will be required, not alone to translate into practice the gains already won in principle, but also to achieve still further revisions of defective principle.

Most important of these is the requirement that 90 per cent of the labor employed on both PWA and work-relief projects shall come from the relief rolls. On work-relief projects the propriety of such a requirement can scarcely be questioned. They consist chiefly of "made-work," undertaken to relieve those in distress who cannot find normal jobs; compensation is in the form of a security wage which is, in effect, a substitute for a dole; the work is planned to absorb a large amount of unskilled labor, handled chiefly by force account. Being a substitute for direct relief, such projects may well draw 90 per cent of their workers from relief rolls.

BUT the PWA projects present a wholly different situation. These are normal public works, designed to create long-term values for the nation, the states or the municipalities they will serve; they involve the installation of materials and equipment, with the attendant employment of many skilled workers; for the most part they will be built under competitive contract making some contractor responsible on the basis of an agreed price, for quality, performance and speed. Employment thus created will be normal employment through normal channels and will pay normal wages. Of all the work-relief expenditures, just such work as this will contribute the greatest impetus toward the restoration of normal conditions. Such projects, therefore, should not be hampered by an arbitrary requirement that 90 per cent of the labor must consist of those who happen to be on the relief rolls; those officials, engineers, contractors and others who are responsible for the productive use of the public funds thus entrusted to them by the taxpayers should not be harassed by the complications inherent in such a procedure.

From a practical viewpoint how is an engineer or contractor going to estimate the cost of a job that he must build over a period of months if he is not to have a free hand in selecting his labor? How, for example, is he to forecast whether, six months from now, he will find on the relief rolls from which he must draw, an adequate supply of the type of men he may need at that time to carry on his work? Moreover, what is the sense of introducing these hazards with no object but to keep thousands of worthy and competent workers out of employment just because they are not on relief rolls? To perform efficiently the kind of work that will be needed to plan and build public works of utility and permanent value surely requires other qualifications than simply being in distress and a recipient of public relief.

ALREADY competent contractors who need the work are showing reluctance to bid on some of those contracts because of this requirement. If the administration wishes to make its public construction policy effective and if it would encourage cities and other communities to advance 55 per cent of the cost of non-federal projects and thereby stimulate normal employment, it should at once rescind a requirement that surely will obstruct such action.

The employment regulations to govern normal public works construction, as distinguished from work-relief projects, should be amended to put such employment on the basis of merit and competence rather than of distress; they should have as their objective the relief of unemployment rather than merely a reduction, for the time being, in the number of public dependents on the relief rolls. Thus only can the work-relief program make the most of its excellent opportunity to contribute toward the relief of unemployment as well as toward the relief of distress.

Willard Chevalier

ENGINEERING NEWS-RECORD
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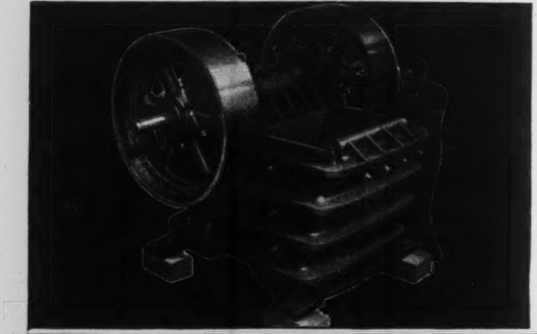
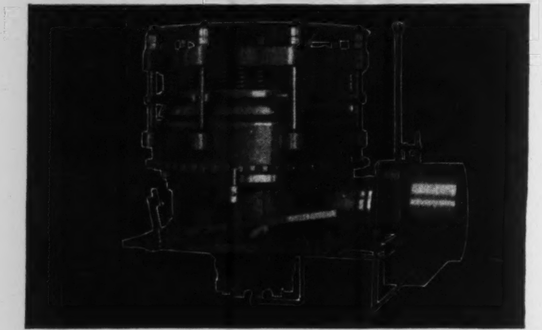


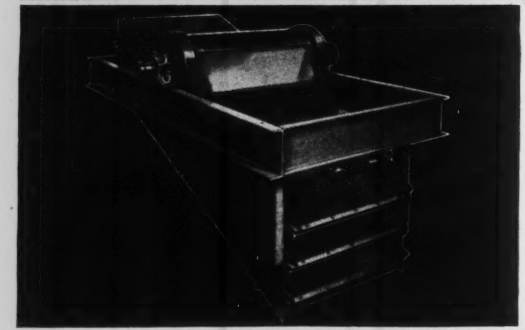
• With its compact but massive steel structure, cylindrical roller bearings and high speed crushing action, the Telsmith-Wheeling Jaw Crusher is ideal for coarse crushing. The roller bearings and higher speed almost double the capacity without any greater expenditure for power. Simple adjustment allows wide sizing range. Up-keep is reduced to a minimum. Write for Bulletin W-2.

## WHEELING JAW CRUSHER—FOR COARSE CRUSHING



• The Telsmith Gyrasphere takes the trouble out of secondary crushing. Working at choke feed, it turns out a bigger tonnage and more cubical product—crushes finer, with low power consumption and up-keep. The reasons—spring relief, rotary head support, spherical head, unit spring design, anti-friction thrust bearings, pressure lubrication, different distribution of crusher pressures. Write for Bulletin Y-2.

## GYRASPHERE CRUSHER—FOR FINER CRUSHING



• The Telsmith Pulsator screens crushed rock, sand, gravel, ore or coal . . . wet or dry. Its circular movement produces a maximum screening action, uniform on every inch of the wire, on every deck, under any load. The toughest alloy steels, the finest anti-friction bearings and special labyrinth and piston ring steels (to protect working parts) give longer life and lower up-keep. Write for Bulletin V-2.

MC-1

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## In the News:

San Francisco is digesting a new report on the desirability of some trunk-line subways, to supplement its present transit properties, which also are recommended for reorganization into a unified system. The report by Robert Ridgeway and Alfred Brahdy of New York is a review of rapid-transit plans prepared by the city public utilities commission.

Rules and Regulations for highway and grade-crossing elimination work were approved last week by the President. Procedure is practically the same as formerly, all plans originating with state highway departments and requiring approval of the district engineer of the Bureau of Public Roads and state representatives of the WPA and the National Emergency Council before being sent to the bureau office at Washington.

Mexico has approved extensive flood-control plans for the lower Rio Grande Valley, to match the work already done on the Texas side of the river. Several dams and an irrigation development are contemplated.

PWA Participation in the relief-works program will be governed by rules believed to be an improvement over old PWA regulations. Increased responsibility for state PWA executives, simpler contracts, advance payment of part of the grants are among the promising items listed.

Mapping Work—37,440 square miles for new maps and 38,320 square miles for revised maps—is contemplated by the Corps of Engineers if an application for $2,000,000 of relief-works money is granted. The work would supplement that of the regular mapping agencies—the Geological Survey and the Coast & Geodetic Survey.

The PWA Housing Program was dealt a new blow on Monday, when the U. S. Circuit Court of Appeals at Cincinnati ruled that the federal government has no power to condemn land for housing purposes. PWA accordingly will acquire land by direct purchase or seek the aid of cities in condemnation suits, as has already been done in New York City.

After Three Years, during which ten acres of new sedimentation and aeration tanks on Ward's Island have stood idle, New York City is to start work again on its 180-m.g.d. sewage plant. Bids are out for the two inlet tunnels under the river and for grit chambers.

## In This Issue:

The Last Bottlenecks in two shore highways north and south of Boston have been removed by the construction of new bridges over the Saugus and Fore Rivers. Exceptionally wide bascules are a part of the projects, each of which includes numerous structural details of interest.

In the Land of Cotton, concrete pavement slabs are being cured by cotton mats in place of wet earth or ponded water. Texas experience indicates that the cost is no more and probably less, and that there are other advantages.

Unusual Opportunity to improve its facilities was afforded the Bureau of Lighthouses by the PWA appropriation of two years ago. On the Great Lakes several modern lighthouses were built incorporating some new ideas in substructure design. These are presented by Charles A. Park, chief engineer, Bureau of Lighthouses.

Limestone and Marl, as well as fine sand, are being handled by hydraulic dredges in building some of the 66 miles of levees along the south shore of Lake Okeechobee in Florida. The hydraulic-fill levees demonstrate advantages over bucket-placed levees and are in general less costly.

Cellular-Steel Sheetpile breakwater construction and design presented interesting problems in connection with a new structure in Calumet Harbor at the southern tip of Lake Michigan. Their solution is presented and analyzed by T. L. Condron, who was engaged as a consultant on the work by the Corps of Engineers.

Floods in midwestern New York State last week caused unprecedented damage in many localities. A report, following a three-day visit to the flooded area by one of the editors, presents the major facts of engineering significance and the important hydrological and meteorological data in so far as they are now available.

## Coming Articles:

A Parkway from Canada to the Gulf, following the mountain crests of the Appalachians for much of its thousand-mile length, is being surveyed at the instance of federal officials. Construction of much of its length is far in the future, but already rapid progress is being made in Shenandoah National Park in Virginia and in the Great Smoky Mountains along the Tennessee-North Carolina boundary. Both the tangible and the dream phases of this great scenic highway will be presented in an article to be published next week.

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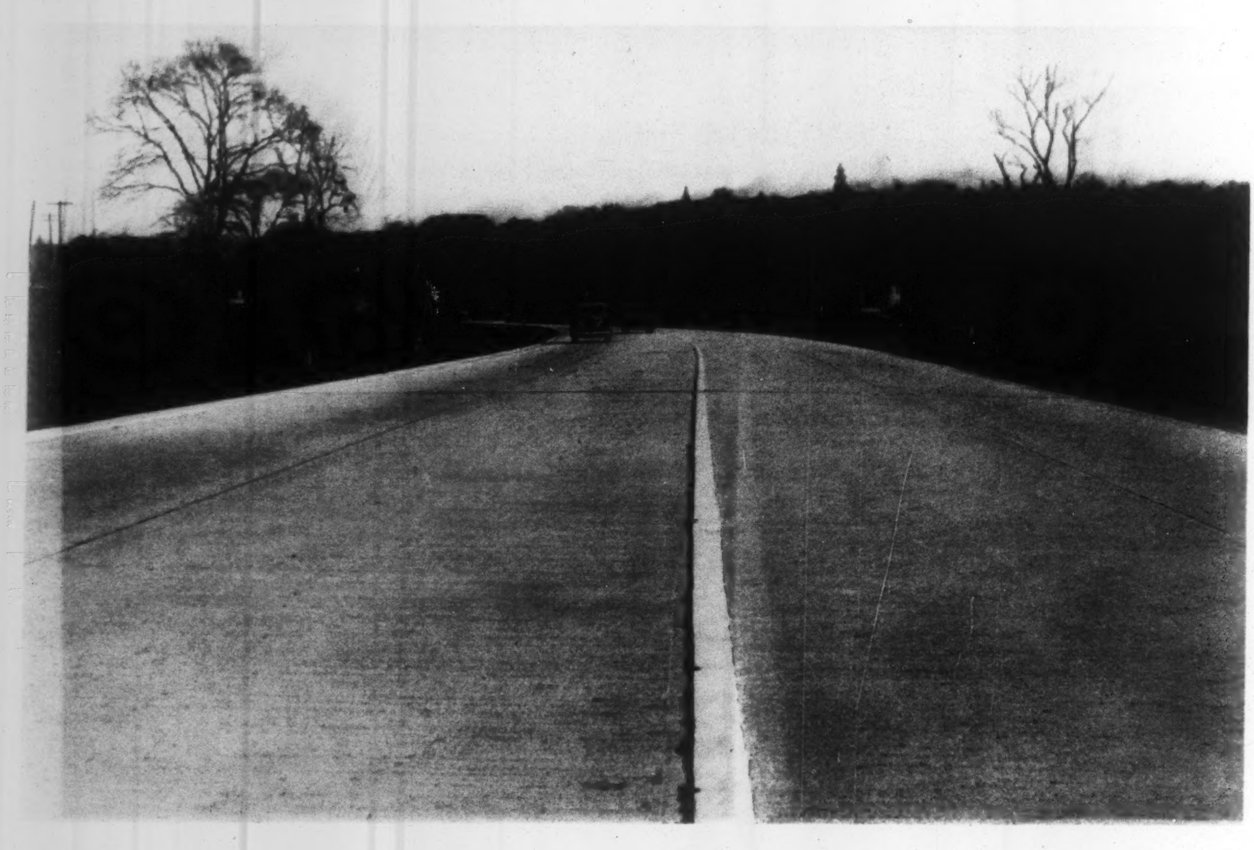
## ENR INDEX—First Half of 1935—IN THIS ISSUE

# why not SAVE the money you're spending on marker maintenance?



This maintenance-free Atlas White marker, installed in concrete pavement on Central Westchester Parkway in White Plains, N. Y., was built by A. W. Banko, Inc., Hastings-on-Hudson, N. Y., for Westchester County Park Commission. It is 6 inches wide, 1 inch deep, and permanently white.

EVERY dollar spent to replace temporary pavement markers—which traffic quickly wears away again—is a dollar that could be saved.

For the cost of one year's maintenance of temporary traffic markers on a busy street, *permanent* white markers that never need replacement can be installed.

***Markers BUILT White***

These permanent markers are made of white concrete. They are *built into* the pavement, as an integral unit, and they last as long as the pavement lasts. Installation in new concrete, or brick, or asphalt pavement is a simple job. It is equally simple to place these permanent white markers in old brick or old asphalt pavements.

***Markers STAY White***

The white surface of these solid white concrete markers is dense, hard and stain-resistant. Neither weather nor traffic can wear these markers away, or blot them out, or erase their permanent whiteness.

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Once these indelible white concrete markers are installed, the expense of marker maintenance is ended. Their first cost is the last cost. And every year after they are placed you save all the money that would have to be spent if temporary markers had to be maintained. Full information will be sent promptly—write Universal Atlas Cement Co. (*United States Steel Corporation Subsidiary*), 208 South La Salle Street, Chicago.

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## ATLAS WHITE TRAFFIC MARKERS

**Made with Atlas White Portland Cement—Plain or Waterproofed**

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July 18, 1935

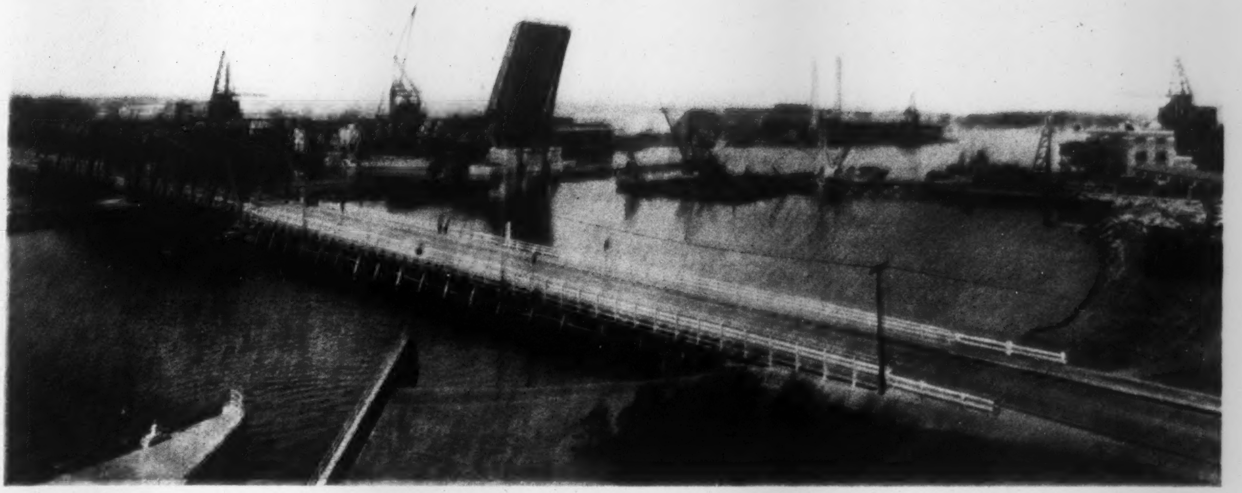

FIG. 1—CONSTRUCTING the new Fore River highway crossing at Quincy, Mass. The double-leaf bascules will provide a 200-ft. opening for boats and a 60-ft. roadway, as compared to a 125-ft. opening and a 30-ft. roadway on the old swing span, which is shown in its new location, to serve as a detour during the construction period.

# New Bridges Remove Bottlenecks on Boston Shore Highways

**Exceptionally wide bascule spans characterize expensive state highway structures over Saugus and Fore Rivers, respectively north and south of the city—Cantilever plate-girder spans, operating equipment for bascules and anchored pier are other features of interest**

TO REMOVE constrictions in the main highway through metropolitan Boston north to New Hampshire and Maine by the new East Boston Tunnel route and south to the Cape Cod resorts, new and expensive bridges are under construction across the Saugus and Fore Rivers, respectively, about 8 miles north and south of the city limits. The importance of the traffic on the two outlets is indicated by Fig. 2, but its freedom of movement has long been hampered by obsolete and narrow bridges at the river crossings named. Similarly the old bridges had been bothersome to navigation, particularly that at Fore River with its big shipyard directly alongside the bridge. By arrangement for PWA allotments in 1933, the Massachusetts department of public works was able to undertake replacement of the old bridges with new wide bridges of modern design costing something over a million dollars each.

The new bridges express tendencies in modern trunk-highway bridge design. They are of superhighway width and of the monumental structure appropriate to urban location and unusual traffic importance. As bridge designs they have technical importance for their unusually wide and relatively long bascule spans and for one or two particulars of substructure design. Similarity of crossing conditions at the two locations also introduced the interesting feature of identical types of bridges with differences of structure, as for example truss spans at Fore River and plate-girder spans at Saugus River. These comparisons and contrasts are interestingly shown by Figs. 3 and 4.

At both bridge locations the streams were bordered by flats not much higher than high-tide level and rising to high ground over widths of 1,200 to 1,500 ft. Both streams carry shipping (that at Fore River being the more important), a considerable part of which is in low-clearance craft. These conditions set the requirements for relatively long approaches and channel-span heights which would clear the majority of the low-clearance river traffic. In addition, the crossing of salt-water streams made it desirable to keep the steelwork well above water level. The requirements for pleasing appearance have been explained. Based on all these conditions similarity of bridge type was a natural decision.

To accentuate the roadways as main-purpose elements of profile, deck struc-

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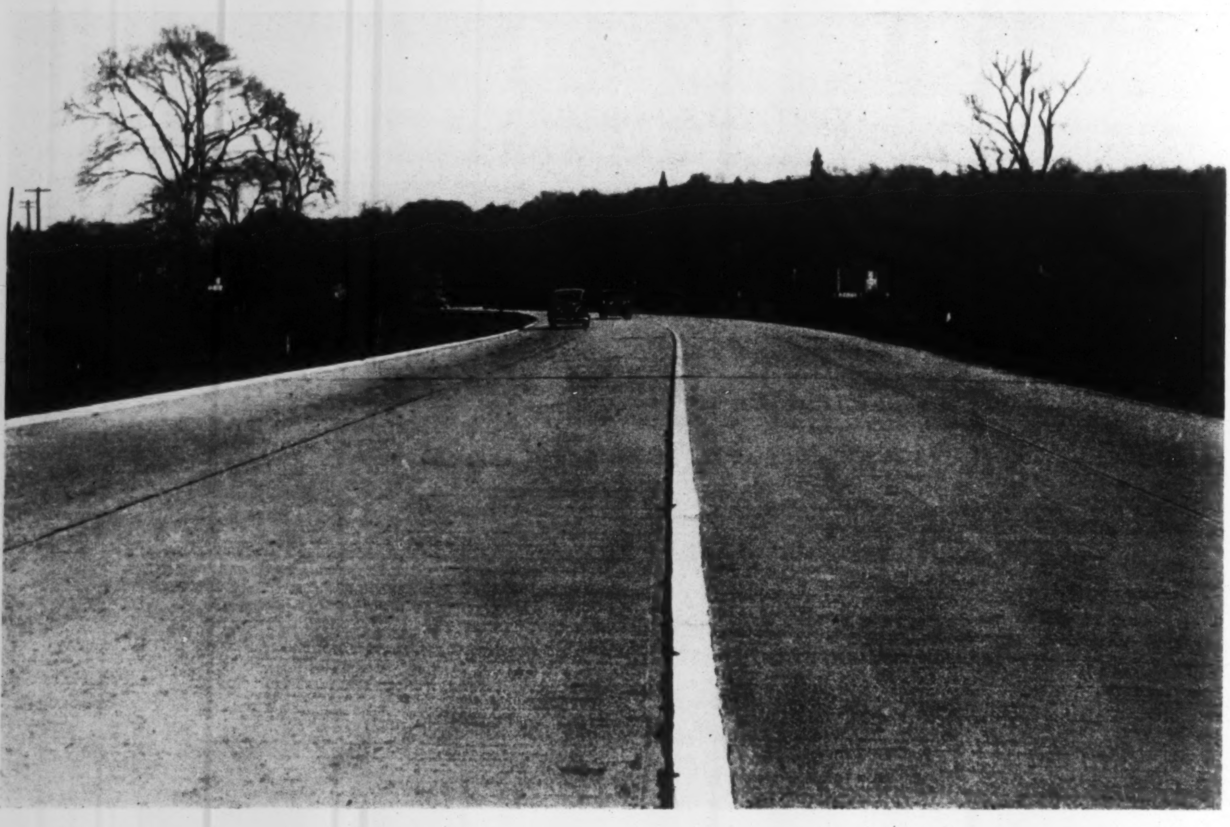

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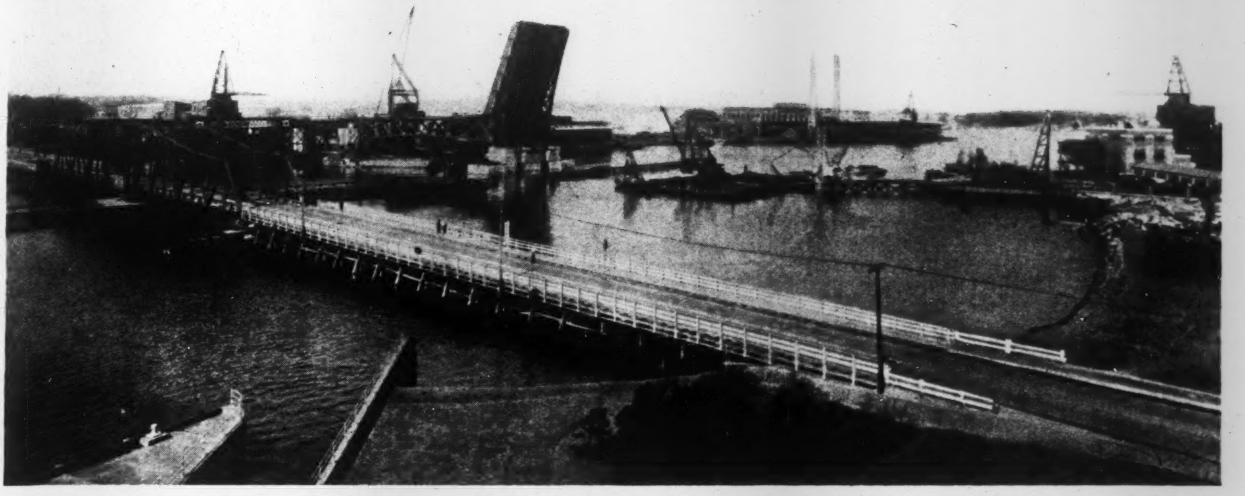

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To accentuate the roadways as main-purpose elements of profile, deck structures were adopted; as shown by the elevations (Figs. 3 and 4) roadways are uninterrupted visually as well as structurally by superstructure. The deck profiles rise gradually from opposite shores to long-radius vertical curves, giving liberal clearance heights of channel spans. At Fore River the ground conditions made symmetrical span arrangement possible; at Saugus River a longer approach was necessary on one side of the channel span, requiring two additional spans, but in all other respects the arrangement is symmetrical. The bottom-chord profiles are alike in both bridges; in both, the piers terminate in massive pylons on each face of the bridge.

The conditions, similar at both bridges, indicated bascule channel spans. By their selection, span profiles would harmonize with the approach span profiles, erection was possible without obstructing free movement of navigation, and obstructing center pier and protection fender were eliminated. In cost the comparison also favored bascule spans. With a 60-ft. roadway, swing spans would have required center piers of nearly as great diameters, which, with the enormous fenders, would have increased the span cost over that of either a bascule or a lift span. At Fore River estimates indicated about the same costs for a bascule or a lift span; at Saugus River they made a lift span more expensive. As between trunnion-type and rolling-lift-type bascules the estimates favored the rolling-lift, which was adopted.

In other general features only two call for special notice. Except for the three long spans at Fore River, which are trusses, all spans are plate girders. This design gave desired limited depth of steel and character of framing to resist salt-water action, a requirement which, it was felt, justified some sacrifice of economic span length. The second point of interest arises in respect to the higher clearances sought for the channel spans; at Fore River, with present river traffic, it is estimated that the new clearance will reduce openings in the ratio of 1 to 7. At both bridges the higher clearance contributed to appearance by making it possible to place the counterweights for bascule operation below deck level and out of sight.

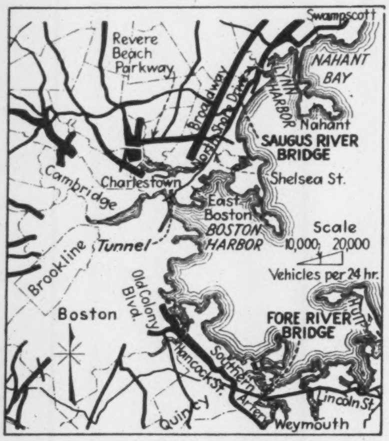

Swampscott
Revere Beach Parkway
NAHANT BAY
Broadway
North Shore Drive
LYNN HARBOR
Nahant
SAUGUS RIVER BRIDGE
Shelsea St.
Charlestown
Cambridge
East Boston
BOSTON HARBOR
Scale
10,000 20,000
Vehicles per 24 hr.
Tunnel
Brookline
Old Colony Blvd.
Boston
FORE RIVER BRIDGE
Hancock St.
Southern Artery
Lincoln St.
Quincy
Weymouth

FIG. 2—TRAFFIC MAP of roads leading out of Boston, showing location of new bridges at Fore River and Saugus River.

Specific details of structure and construction are better described by considering the bridges separately, with a rangement of the drawings to contra similar parts of the two structures.

## Fore River Bridge

Located on the main road south fro Boston to Cohasset, Hull and Cape Co the old Fore River crossing betwe Quincy and Weymouth, with its widt of only 29 ft., had long constituted serious bottleneck. In addition, th channel span was of swing type, an the center pier was an obstacle to boat particularly the larger ones that use th river in considerable numbers when th Fore River shipyard of the Bethlehe Steel Co. is busy. These existing con ditions had an important bearing on th design of the new bridge, dictating th use of a 60-ft. roadway and a 219-ft bascule-type channel span providing a clear opening of 175 ft. as against tw 100-ft. openings on the old swing span

The new bridge is on the same align ment as the old bridge, which wa moved downstream to serve as a detour during the construction period (*ENR*, Dec. 20, 1934, p. 785). Including re taining-wall approaches, the new con struction has a length of 2,414 ft. On either side of the bascule span there is a 212-ft. truss span and three steel girder spans of about 90 ft. each. The truss spans contain five lines of deck trusses 18 ft. on centers. Rolled-section floor beams are framed into the top chords and support I-beam stringers carrying a concrete deck (Fig. 5). The girder approach spans are similar but with five lines of girders. In addition to the 60-ft. roadway, there is an 8-ft. sidewalk on either side of the deck.

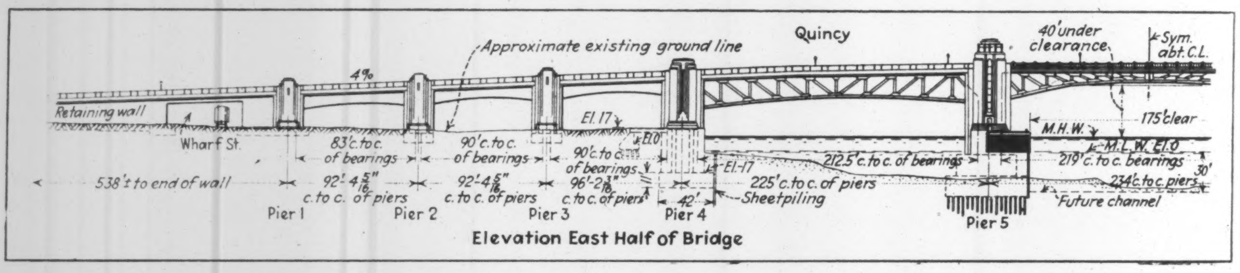

Approximate existing ground line
Quincy
40' under clearance
Sym. abt. C.L.
4%
Retaining wall
Wharf St.
El. 17
El. 0
175' clear
M.H.W.
M.L.W. El. 0
83' c. to c. of bearings
90' c. to c. of bearings
90' c. to c. of bearings
El. -17
212.5' c. to c. of bearings
219' c. to c. bearings
30'
538' ± to end of wall
92'-4 5/16" c. to c. of piers
92'-4 5/16" c. to c. of piers
96'-2 3/16" c. to c. of piers
42'
225' c. to c. of piers
234' c. to c. piers
Sheetpiling
Future channel
Pier 1
Pier 2
Pier 3
Pier 4
Pier 5
Elevation East Half of Bridge

FIG. 3—FORE RIVER BRIDGE ELEVATION, symmetrical except for approach fills about bascule span. Main spans are trusses.

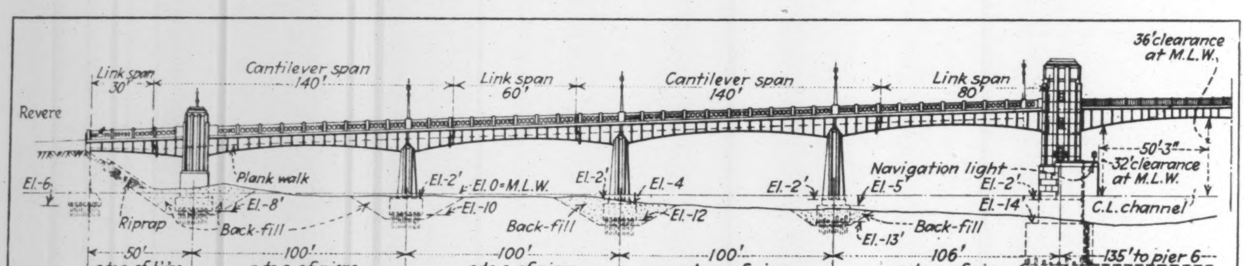

36' clearance at M.L.W.
Link span 30'
Cantilever span 140'
Link span 60'
Cantilever span 140'
Link span 80'
Revere
50'-3"
32' clearance at M.L.W.
Navigation light
Plank walk
El. -6
El. -2'
El. 0 = M.L.W.
El. -2'
El. -4
El. -2'
El. -5'
El. -2'
El. -14'
C.L. channel
El. -8'
El. -10
El. -12
Riprap
Back-fill
Back-fill
Back-fill
El. -13'
50' c. to c. of brgs.
100' c. to c. of piers
100' c. to c. of piers
100' c. to c. of piers
106' c. to c. of piers
135' to pier 6
El. -34'
Cable at El. -34'
Pier 1
Pier 2
Pier 3
Pier 4
Pier 5
Elevation South Half of Bridge
North half same except for five instead of three 100' spans

FIG. 4—SAUGUS RIVER BRIDGE ELEVATION, showing all-plate-girder structure and cantilever-span arrangement.

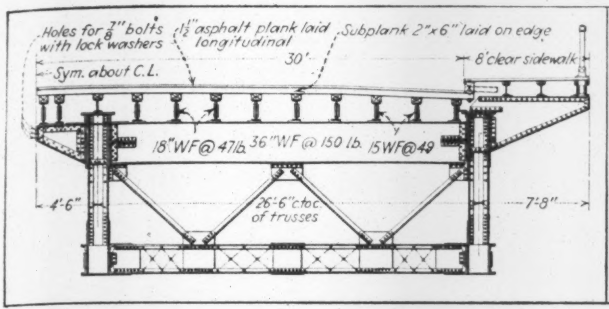

Holes for 7/8" bolts with lock washers
1½" asphalt plank laid longitudinal
Subplank 2"x6" laid on edge
30'
8' clear sidewalk
Sym. about C.L.
18" WF @ 47 lb.
36" WF @ 150 lb.
15 WF @ 49
4'-6"
26'-6" c. to c. of trusses
7'-8"

FIG. 5—HALF CROSS-SECTION of Fore River Bridge at bascule span, showing bolt coupling to twin half section.

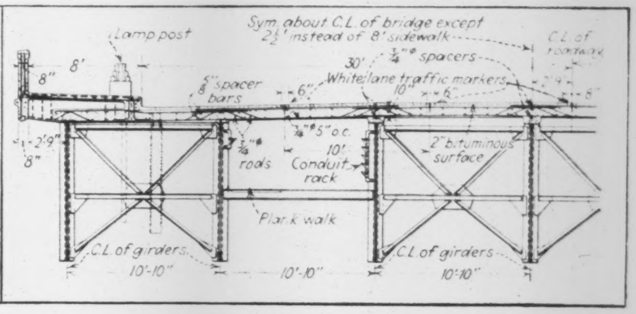

Lamp post
Sym. about C.L. of bridge except 2½' instead of 8' sidewalk
C.L. of roadway
8'
8"
5/8" spacer bars
30'
¾"ø spacers
White lane traffic markers
10"
6"
2'-9"
rods
¾"ø 5" o.c.
10'
Conduit rack
2" bituminous surface
Plank walk
C.L. of girders
10'-10"

FIG. 6—HALF CROSS-SECTION of Saugus River Bridge at approach span, showing floor slab thickening over girders as substitute for stringers.

*Substructure* — Foundation material beneath the estuary silt is a firm sand and gravel, which was satisfactory to support all approach piers at a relatively shallow depth. However, on the bascule span piers, both of which are offshore, piling was necessary because of the heavy load concentrations. Piling was also utilized under an ingenious anchor block (Fig. 7), used in conjunction with approach pier 4 on the Quincy side, but to provide resistance to sliding rather than additional load-carrying capacity.

As shown in Fig. 3, pier 4 is at the water's edge, and the anchor block assists in stabilizing it against overturning. Located some 25 ft. back of the pier, the anchor block is connected to the pier by a reinforced-concrete slab 18 in. thick. Design studies indicated that this scheme of using an anchor block was considerably cheaper than enlarging the base of the pier or founding it on piles, to provide the necessary factor of safety.

*Bascule Span* — Principal technical interest in the Quincy-Weymouth bridge centers in the bascule span. Although of common under-floor-counterweight type, it is notable for large-plan dimensions, about 288x77 ft., including tail deck and sidewalks. By dividing this area along both its transverse and longitudinal centerlines, four leaves result, each 144 ft. long (109½ ft. c. to c. of trunnions) and 26½ ft. wide between main trusses. Under normal conditions the two leaves, side by side, are fastened together and operate together. The advantage in making them structurally separate lies principally in the fact that one-half of the bridge can be closed for repairs without completely interrupting traffic.

The details of these bascule leaves are shown in Fig. 8. From a depth of about 6 ft. at the end or center lock, the trusses increase to about 20 ft. at the trunnion. Chords and web members are built up of angles and plates, the bottom chords braced apart by a square laced-angle member, and the top chords by the floor beams, which are 36-in., 150-lb. girder beams. Stringers set on top of the floor beams are 18-in. I-sections, which carry nailing strips for the creosoted timber floor. Asphalt plank comprises the wearing surface. The bridge is designed for an H-20 loading. Sidewalks are carried on 7 ft. 8 in. cantilever brackets, while similar cantilevers 4½ ft. long support the deck stringers along the centerline of the bridge in the space between the interior trusses of the adjacent leaves.

*Bascule Operating Machinery*—The division of the bascule span into four leaves with possibility of having to operate any leaf separately introduced features of special interest in the operating machinery.

Each leaf is balanced for its own dead weight of 825 tons by a concrete counterweight weighing 1,000 tons,

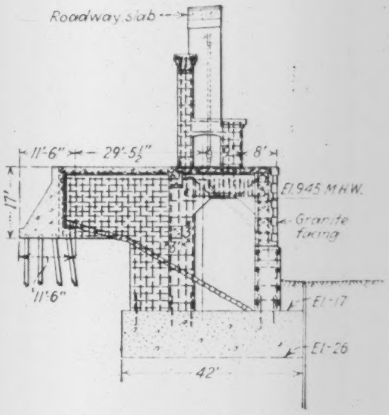

Roadway slab
11'-6"
29'-5½"
8'
17'
El. 9.45 M.H.W.
Granite facing
11'-6"
El.-17
El.-26
42'

FIG. 7—ANCHOR BLOCK and section of pier No. 4 of Fore River Bridge as indicated on Fig. 3.

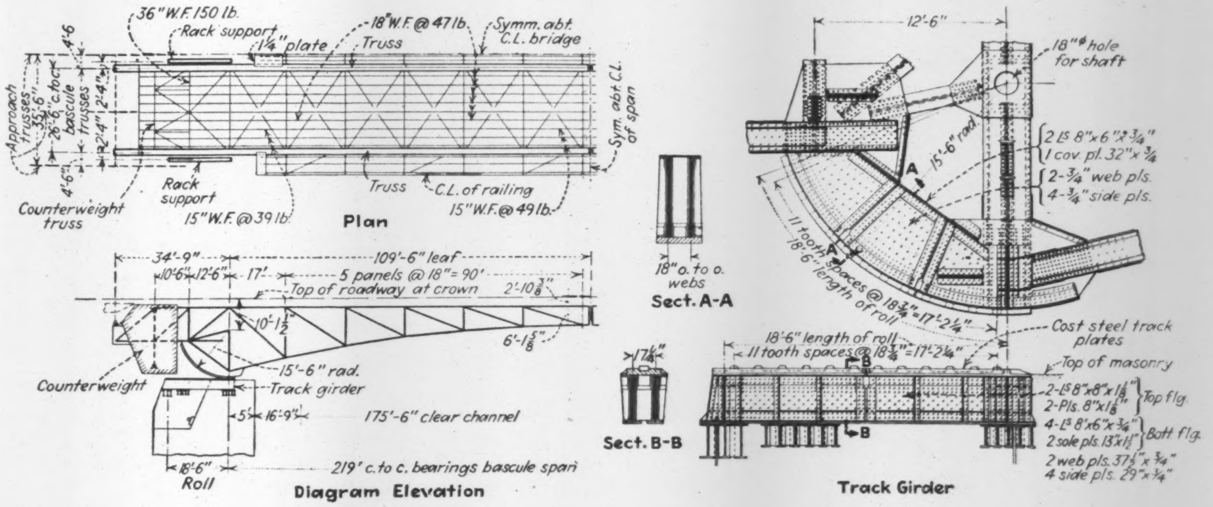

36" W.F. 150 lb.
Rack support
1¼" plate
18" W.F. @ 47 lb.
Truss
Symm. abt. C.L. bridge
Approach trusses 35'-6"
26'-6" c. to c. bascule trusses
4'-6"
2'-4"
Sym. abt. C.L. of span
Rack support
Truss
C.L. of railing
15" W.F. @ 49 lb.
Counterweight truss
15" W.F. @ 39 lb.
Plan
34'-9"
109'-6" leaf
10'-6"
12'-6"
17'
5 panels @ 18" = 90'
Top of roadway at crown
2'-10⅜"
10'-1½"
6'-1⅝"
15'-6" rad.
Track girder
Counterweight
5'
16'-9½"
175'-6" clear channel
18'-6" Roll
219' c. to c. bearings bascule span
Diagram Elevation
18" o. to o. webs
Sect. A-A
1⅞"
Sect. B-B
12'-6"
18"ø hole for shaft
15'-6" rad.
2 Ls 8"x6"x¾"
1 cov. pl. 32"x¾
2-¾" web pls.
4-¾" side pls.
11 tooth spaces @ 18⅞" = 17'-2¼"
18'-6" length of roll
Cast steel track plates
Top of masonry
2-Ls 8"x8"x1⅛"
2-Pls. 8"x1⅛"
Top flg.
4-Ls 8"x6"x¾"
2 sole pls. 13"x1½"
Bott flg.
2 web pls. 37½"x¾"
4 side pls. 29"x¾"
Track Girder

FIG. 8—BASCULE-SPAN PLAN (left) and roller girder and track details (right) for Fore River Bridge.

Four sets of machinery are provided, one for each half leaf, located on the moving part. Each set is operated by two 50-hp. motors geared to two operating pinions, which engage fixed racks attached to structural-steel supports from the bascule pier. These motors will operate the leaf either up or down in 1¼ min. Means are also provided for operation by hand in case of emergency. One motor-mounted, spring-set, thrustor-type shoe brake is provided for each motor, and one emergency brake of the same type, except that it is floor-mounted, is provided for each half leaf. Horizontal swinging gates driven by electric motors are provided to control traffic, located about 100 ft. from each end of the bascule span. Near these gates are placed three-color, one-way, traffic-control signals.

Alternating current (3 phase, 60 cycle, 440 volt) is provided for operating the leaves. The main motors are of the wound rotor hoist type, fully enclosed. Normally both motors on each set of machinery are in service, but either motor can handle its half leaf in case of emergency. A selector switch is provided so that any combination of motors desired can be used. One motor can be cut out of each half leaf if desired, or both motors can be cut out of one half leaf, and the other half leaf operated independently. Both leaves are operated from one operator's room located in an end tower. The main control board together with the resistors required in connection with the control are located in the switchboard room below the operator's room. The main control bench is located in the operator's room on the channel side. It contains two duplex reversing-drum controllers, one for each leaf, emergency brake-control switches, voltmeter, ammeter, position indicators, indicator lights, switches for lights and other circuits, sealed switches for bypassing in an emergency, and certain sequence interlocks in the controls. A separate control bench for operating the traffic gates is located on the roadway side of the operator's room.

A limit switch is provided for each leaf, to cut off the power from the leaf motors and set the motor brakes near each end of the leaf travel. The control is so arranged that, after the leaf has passed through the limits near each end of its travel, reduced power is made available for further operation of the leaf by returning the controller handle to the "off" position. The gate motors are also equipped with limit switches. The controllers and switches for operating the span, gates and traffic signals are so interlocked that they can be operated only in proper sequence. When raising, the signals are set first, then the gates closed, then the leaves raised. When lowering, the sequence is reversed.

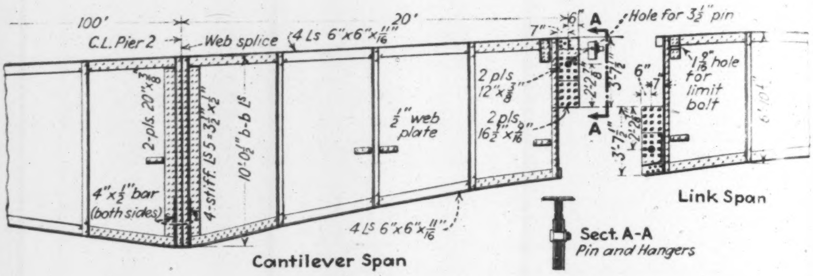

100'
C.L. Pier 2
Web splice
20'
4 Ls 6"x6"x$\frac{11}{16}$"
7"
6"
A
Hole for 3½" pin
2-pls. 20"x$\frac{5}{8}$"
4-stiff. Ls 5-3½"x$\frac{5}{8}$"
10'-0½" b-b Ls
½" web plate
2 pls 12"x$\frac{3}{8}$"
2 pls 16½"x$\frac{9}{16}$"
6"
1$\frac{9}{16}$" hole for limit bolt
4"x½" bar (both sides)
4 Ls 6"x6"x$\frac{11}{16}$"
Cantilever Span
Sect. A-A
Pin and Hangers
Link Span

FIG. 10—GIRDER DETAILS for cantilever spans of Saugus River Bridge.

## Saugus River Bridge

The new bridge north of Boston over the Saugus River on the road to Swampscott and Lynn provides contrast to the Quincy-Weymouth bridge in that plate girders are used exclusively in the bascule as well as the approaches and in the fact that extensive use of pile foundations was necessary. Also, approach girders are arranged in a series of cantilever and link spans as compared to the simple spans on the Quincy-Weymouth structure. The bascule span of the Saugus River Bridge is composed of four leaves, as on the Quincy-Weymouth bridge, but each leaf is only 81 ft. long (63 ft. c. to c. of trunnions) as compared to 144 ft. on the latter structure.

The Saugus River Bridge eliminates the last bottleneck (a 26-ft.-wide bascule span) on the main highway known as Shore Road, which connects, via wid-

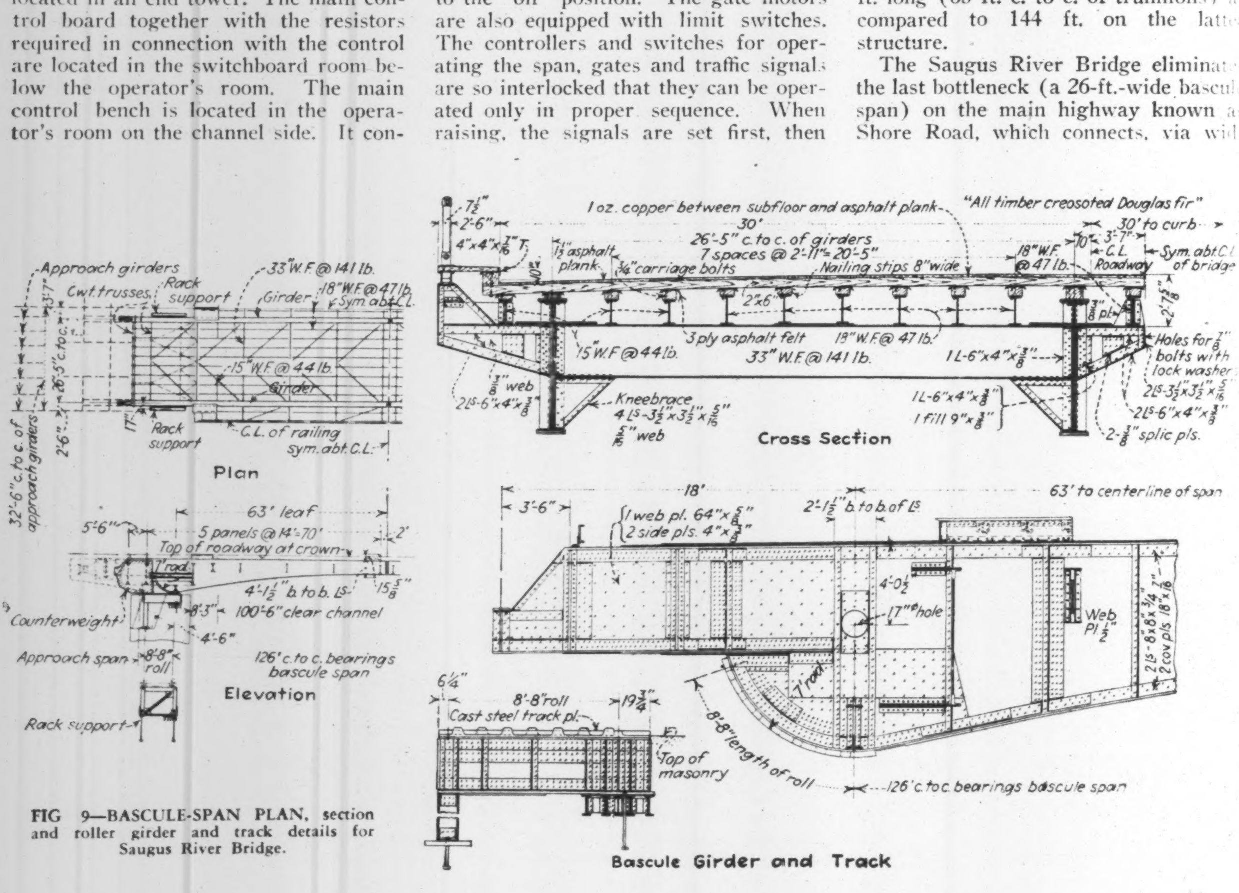

Approach girders
Cwt. trusses
Rack support
33" W.F. @ 141 lb.
Girder
18" W.F. @ 47 lb.
Sym. abt. C.L.
15" W.F. @ 44 lb.
Girder
26'-5" c. to c.
32'-6" c. to c. of approach girders
Rack support
C.L. of railing sym. abt. C.L.
Plan
63' leaf
5 panels @ 14'=70'
Top of roadway at crown
4'-1½" b. to b. Ls
100'-6" clear channel
Counterweight
Approach span
8'-8" roll
126' c. to c. bearings bascule span
Elevation
Rack support
1 oz. copper between subfloor and asphalt plank
"All timber creosoted Douglas fir"
30'
26'-5" c. to c. of girders
7 spaces @ 2'-11"=20'-5"
30' to curb
C.L. Roadway
Sym. abt. C.L. of bridge
1½" asphalt plank
¾" carriage bolts
Nailing strips 8" wide
18" W.F. @ 47 lb.
3 ply asphalt felt
19" W.F. @ 47 lb.
15" W.F. @ 44 lb.
33" W.F. @ 141 lb.
1 L-6"x4"x$\frac{3}{8}$"
Holes for $\frac{7}{8}$" bolts with lock washers
Kneebrace
4 Ls-3½"x3½"x$\frac{5}{16}$"
$\frac{5}{16}$" web
2 Ls-6"x4"x$\frac{3}{8}$"
1 fill 9"x$\frac{3}{8}$"
2-$\frac{3}{8}$" splic pls.
Cross Section
18'
3'-6"
1 web pl. 64"x$\frac{5}{8}$"
2 side pls. 4"x$\frac{5}{8}$"
2'-1½" b. to b. of Ls
63' to centerline of span
4'-0½"
17" hole
Web Pl ½"
8'-8" roll
19¾"
Cast steel track pl.
Top of masonry
8'-8" length of roll
126' c. to c. bearings bascule span
Bascule Girder and Track

FIG 9—BASCULE-SPAN PLAN, section and roller girder and track details for Saugus River Bridge.

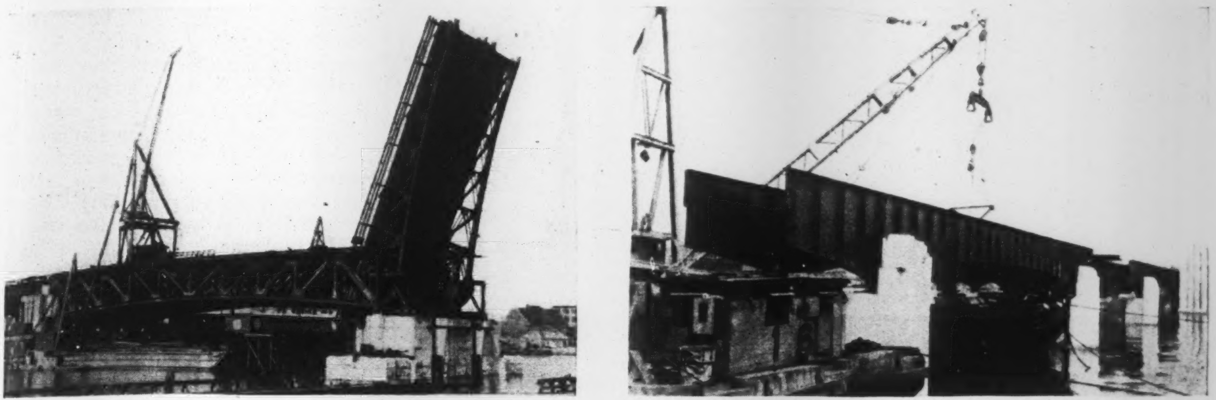


FIG. 11—CONSTRUCTION VIEWS of new Boston Shore highway bridges, over Fore River (left) and over Saugus River (right).

ened streets in East Boston, with the new vehicular tunnel into Boston. An improved south approach to the new bridge is also a part of the current project. In this south approach Shore Road and Lynnway (a parallel pleasure-traffic boulevard) converge. In the old arrangement this intersection occurred in a wide plaza where traffic movements were uncertain and dangerous. Also, Shore Road crossed the tracks of the Boston, Revere Beach & Lynn Railroad just before it entered this plaza. The new layout includes a grade-separation at this point (a four-span continuous steel-girder structure), which not only eliminates the railroad crossing but also any crossing of traffic lanes between the two highways.

*Substructure*—The topography at the bridge site permitted the new bridge to be built alongside the old without disturbing traffic or requiring it to detour. Foundation material consisted of layers of silty sand and shells overlying soft blue clay extending downward to coarse sand or gravel at El. —90 to —100. All pier bases were founded in the clay (which had a bearing value of only about 1,000 lb. per sq.ft.) on timber piles varying from 50 to 100 ft. Elevation of the bases of the approach piers varied from El. —8 to —15, while the bascule piers were founded at El. —34. Twelve piers and abutments required 27,700 ft. of 100-ft., 18,000 ft. of 95-ft., 56,000 ft. of 90-ft., 58,000 ft. of 85-ft. and 26,000 ft. of 80-ft. piles, as well as small footages of shorter piles. As on the Quincy-Weymouth bridge, all foundations were faced with granite.

*Approach Spans* — Details of the bridge supertructure are given in Figs. 6, 9 and 10. Within a 72-ft. over-all width, a 60-ft. roadway, an $8\frac{1}{2}$-ft. and a $2\frac{1}{2}$-ft. sidewalk are provided. In the approach spans there are seven lines of plate girders with $\frac{7}{16}$-in. webs, $6x6x\frac{5}{8}$-in. flange angles and coverplates as required. These girders are arranged in groups continuous over two piers, with suspended or link spans between. Thus the continuous girders are 140 ft. long, covering a 100-ft. span and cantilevering 20 ft. beyond the piers. Link spans vary in length from 30 to 80 ft. The girders are 10 ft. deep over the piers and about 6 ft. deep at the center. The connection between the cantilever and link spans is by means of vertical hangers with $3\frac{1}{2}$-in. diameter piers at either end (Fig. 10).

The roadway of the approach spans is somewhat unusual in that no steel stringers are used. The 10-in. concrete deck slab is haunched over each line of girders and reinforced to provide the necessary strength. Clip angles along the top chord of the girders afford a positive bond with the deck slab. The roadway is on a 4 per cent grade south of the bascule span and on a 2.4 per cent grade north of it. The bascule span itself is on a 154-ft. vertical curve of 6,500-ft. radius.

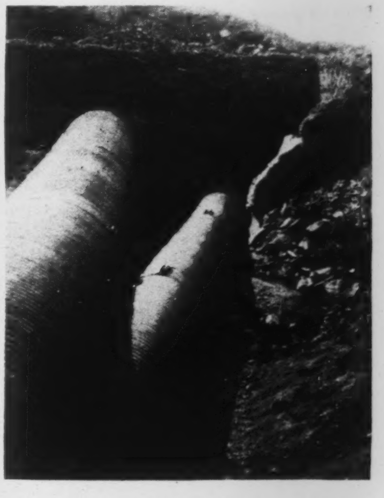
*Bascule Span*—The bascule opening is served by four leaves—two two-leaf bascules side by side (Fig. 9). As on the Quincy-Weymouth bridge, this arrangement is largely dictated by the desire to permit servicing the bridge without shutting it down entirely. Each 81-ft. leaf is made up of two girders spaced 26 ft. 5 in. c. to c. and varying in depth from 4 to 11 ft. Floor beams of 33-in. 141-lb. sections rest on brackets framing into the girders. In all other respects the bascule is similar to the one in the Fore River Bridge.

### Direction

The Fore River bridge was designed by the Massachusetts department of public works and both bridges are being constructed under its direction. A. W. Dean is chief engineer, and G. E. Harkness is bridge engineer. The Saugus River Bridge was designed by J. R. Worcester & Co., Boston, Mass., under the direction of the late Eugene E. Pettee. Keller & Harrington, Chicago, Ill., were consulting engineers on the bascule spans of both bridges. McClintic-Marshall Corp., Bethlehem, Pa., is the superstructure contractor on both bridges. V. Barletta Co., Boston, built the Fore River substructure, and Crandall Engineering Co., Boston, the Saugus River substructure.

# Pants-Leg Pipe Section Merges Two Sewers

THE successful merging of two sewers into a 6-ft. corrugated-iron pipe outlet was recently accomplished by the street department of Topeka, Kan., by means of a special forked pipe, shown in the accompanying picture. In improving the drainage of the Kansas free fair grounds at the capital city, two old sewers of different size had to be joined in a single outlet sewer. A 72-in. corrugated-iron pipe with a heavy asphalt invert paving was adopted for the outlet. The special section of forked pipe effected the double connection of the pipe outlet to the two old sewers with no requirement of skilled labor. The new construction was an FERA operation, with materials supplied by the city.



FORKED SECTION of corrugated-iron pipe connects two old sewers to a joint outlet sewer.

# Cheap Curing with Cotton Mats on Texas Pavements

## Seven trial mats used during 1934 and since have been relaid 57 times and appear good for use 25 times more

By J. G. Rollins
*Division Engineer, Texas State Highway Dept., Austin, Texas*

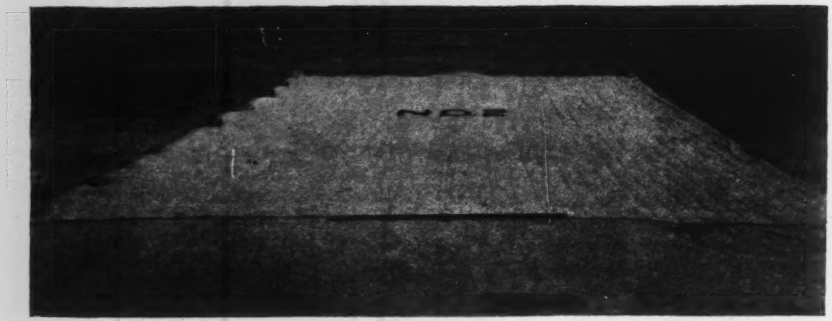


FIG. 1—COTTON CURING MAT in condition for future repeated use after being in service for 57 periods of 72 hours each.

THE SUBSTITUTION of home-land cotton for imported burlap in curing concrete pavement has been vigorously undertaken by the Texas highway department under state highway engineer Gibb Gilchrist. As a first step, taken in 1933, some cotton mats were sent to the Bureau of Public Roads laboratories in Washington for tests to determine their efficiency in curing. Detailed favorable reports on the tests were published in the July, 1933, and November, 1934, issues of *Public Roads.*

To determine the cost of curing with cotton mats for 72 hours in comparison with that of the then-employed ten-day, wet-burlap-wet-earth, and wet-burlap-ponded water methods, seven cotton mats were placed in use by the Texas department along with the other methods in December, 1933, to be used until they wore out. Fig. 1 shows one of these cotton mats that has been in use on six different projects, has been replaced 57 times on pavement for 72-hour minimum periods, and has been saturated with water for a total of 352 days. It is still in good condition, and it appears that it can be used at least 25 times more.

Cotton mats have been used exclusively on several projects in Texas, as shown in Fig. 2. The total cost of curing per square yard of pavement has been approximately 3c. On one project it was 2.62c., figuring the life of the mats at 50 placements and exclusive of the cost of moving them from one project to another. If cotton mats are used extensively, the first cost of the mats will no doubt be appreciably reduced. An arrangement should be worked out whereby they will be used continuously until their life is expended, eliminating as far as possible carrying in storage between projects. Data on compressive strengths of cores cut from pavement cured by the three different methods show higher average strengths for the cotton-mat-cured specimens.

In comparison with the ten-day, wet-burlap-wet-earth and ponded-water methods, data compiled to date show that the advantages of the 72-hour saturated cotton mats are as follows:

1. Pavements have shown a higher compressive strength.

2. In the long run the cotton-mat method has cost no more and probably less per square yard of pavement cured.

3. The curing with cotton mats was more uniform, as a film of water was constantly held over the entire surface of the slab, which could easily be inspected at all times. Where wet burlap was replaced by earth covering and wetted, or by earthen levies and ponded water, there was a period during which the pavement was liable to loss of its moisture, and changes of temperature.

4. Where the subgrade and shoulders were constructed of selected soil, it was not disturbed or contaminated with inferior soil during pavement construction where cotton mats were used.

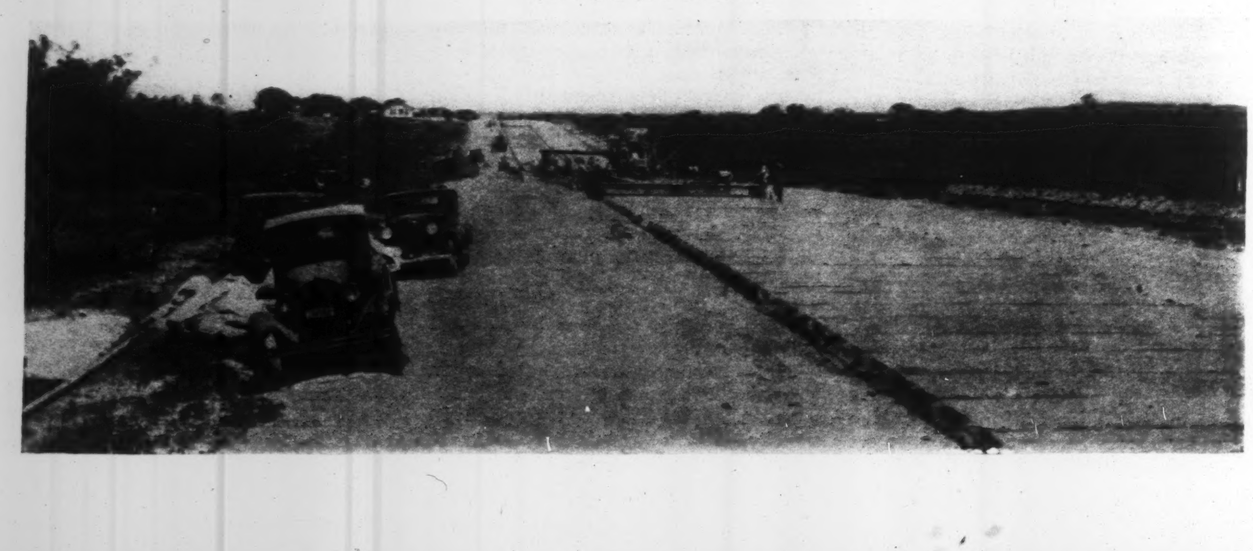
5. Where the concrete was a base for bituminous topping, the scarified surface of concrete was perfectly clean when the topping was applied, and a better bond was obtained between the base and topping.

6. When pavement was laid on city streets where earth for covering had to be hauled in and out, the mats were much more economical.

7. Where ponded water was employed, there was danger of leakage and of over-wetting the foundation soil at concentrated points, which might non-uniformly expand a subgrade soil of high volumetric change and cause irregularities in the pavement surface.

Aside from the efficiency and economy of curing with cotton mats—figuring the life of a mat at 50 placements — approximately one bale of cotton will be expended for every mile of concrete pavement laid and cured with cotton mats.

FIG. 2—TYPICAL paving operation, showing neat clean appearance of curing process when cotton mats are used.



# New Types of Substructure for Great Lakes Lighthouses

## Striking combinations of concrete and cribwork in heavy piers—Adaptation of sheetpiling to shallow water piers —Modern light and control equipment exemplified—Comprehensive renovation and construction program in progress

**By Chas. A. Park**
*Chief Engineer, Bureau of Lighthouses, Washington, D. C.*

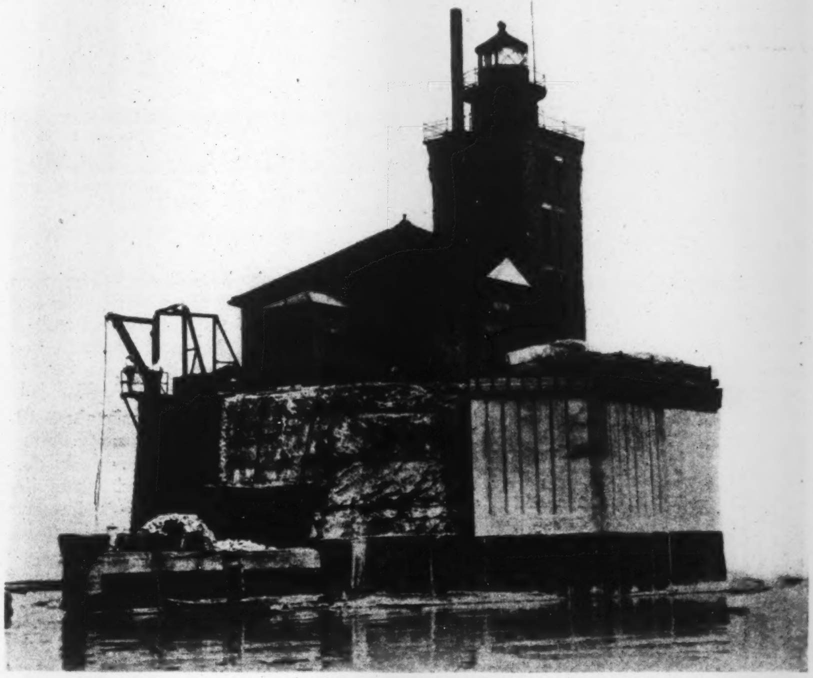


FIG. 1—RENEWAL of ice plating and refacing a damaged pier with reinforced concrete at the Port Austin Reef light station, Lake Huron.

NEW and interesting designs of foundations for soft bottoms have been developed in the recent extensive program of lighthouse construction and renovation financed by PWA funds. Three of these new substructure designs for Great Lakes lighthouses are illustrated. Incidentally the superstructure, mechanical equipment and lantern design of these lighthouses are of the most modern character, some of them being equipped for remote control by radio. To the civil engineer the substructure design is of most interest, and description will be confined largely to the problems there involved.

### General conditions

The foundations illustrated are for new lighthouses on Lake Michigan. All are open-sea structures in depths of 20 to 30 ft., except one located on a reef having a depth of about 10 ft. All called for submarine construction on a sand bottom. Besides provision for wave and current forces and superstructure loads, including wind pressure, severe ice pressure and impact had to be considered as indicated by the armoring of the bearing structure proper shown in the drawings. As indicating possible destructive results of ice impact, attention is called to Figs. 1 and 2.

A substructure of substantial size is indicated by these conditions of ice loading, floe pressure and impact due to current and wave action. Size is required also by the need on sand bottom to keep within the base area the resultant of superstructure dead load and the active overturning forces, including wind pressure on the tall light tower. Operation also calls for landing and servicing space about the tower. The open-sea lighthouse substructure is a problem in submarine pier design and construction, of which the examples illustrated are representative. These examples show the substructures for four lighthouses, two of which are duplicates. The three different designs will be considered separately.

### Square timber crib pier

Two of the Lake Michigan lighthouses replace lightships at Grays Reef and at Minneapolis Shoal. In both locations a good natural bottom was found at depths of 27 and 21 ft., respectively. Borings indicated material of ample supporting power and not likely to offer danger from scouring. Identical structures were possible under the similar conditions and were adopted. The structure at Grays Reef (Fig. 3), is selected for illustration. As shown by the drawings, the essential structural elements are a filled timber crib and a massive reinforced-concrete slab deck or crib cap.

The crib is 64 ft. square with solid outside walls 12x12 in. Oregon fir timbers and interior partitions of the same size timbers were set alternately at right angles. The partitions divide the crib into 49 cells, 24 of which are around the perimeter and are filled with concrete. To permit filling with concrete, the open partition-wall sides of the outside cells are closed by sheet iron, as shown by Fig. 4, dividing the outer ring of cells into filling compartments of three cells each. The interior 25 cells are floored with plank laid with 2-in. spaces between, as shown by Fig. 4; these cells are filled with stone.

Rigid construction was employed. All contacting timbers in side walls and at intersections are drift-pinned; at all angles of partitions with the outside walls are vertical 6x6x½-in. angles bolted through both wall and partition timbers; partitions in the inside three panels are bolted together in pairs through filler timbers; two lines of across-crib tierods one line just inside the outer wall and one line 5½ ft. farther in, bind the crib together horizontally, and four pairs of vertical tierods around the center cell give a similar connection from top to bottom. These tierods act to prevent distortion when the cells are filled with stone and concrete, both during the filling and later.

The filled crib is underwater structure. An overall concrete slab 6 ft. thick and carrying on its edges a heavy concrete wall caps the crib resting on the crib timbers and the concrete fill of the outside cells. Fig. 3 indicates the manner in which the steelwork of house and tower are framed into the slab and surmounting wall. This wall has vertical sides at the corners, and between is alternately a coved panel and a vertical

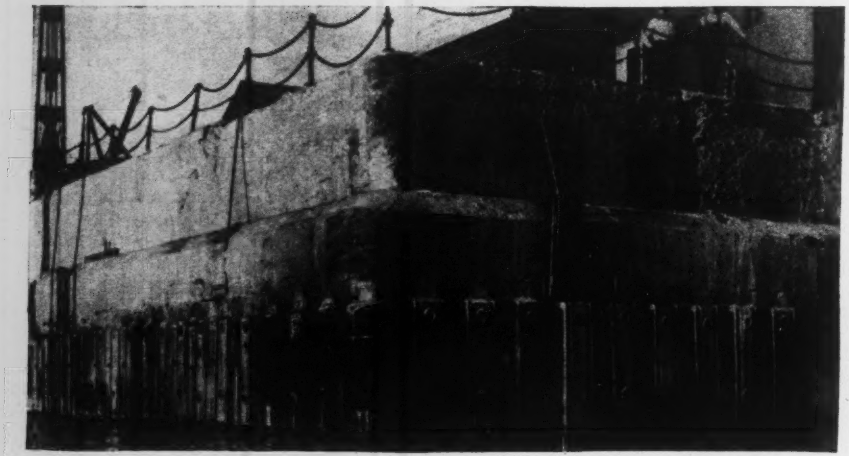


FIG. 4—TIMBER CRIB DETAILS for Grays Reef Lighthouse, showing elaborate piling set in trench drilled in rock, secured by lead-sleeve-type anchors, all cavities being filled with tremie-placed concrete.

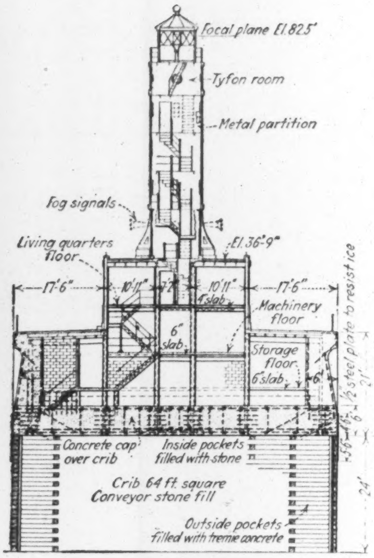

Focal plane El. 82.5'
Tyfon room
Metal partition
Fog signals
Living quarters floor
El. 36'-9"
17'-6"
10'-11"
7'-2"
10'-11"
4" slab
17'-6"
Machinery floor
6" slab
Storage floor
6" slab
6"
½" steel plate to resist ice
21'
5'-6"
4'-6"
6'
Concrete cap over crib
Inside pockets filled with stone
Crib 64 ft. square
Conveyor stone fill
Outside pockets filled with tremie concrete
24'

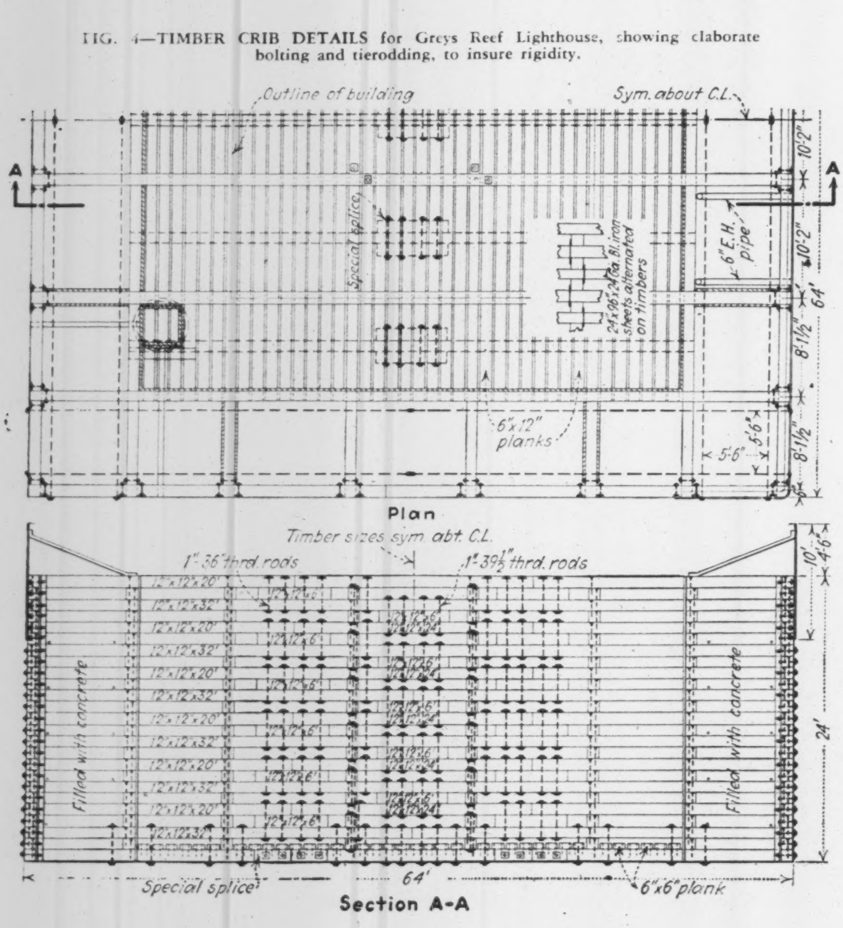
FIG. 3—CONCRETE SUPERSTRUCTURE of Grays Reef Lighthouse rests on 64-ft.-square timber crib which was filled with stone and concrete after being towed to the site. Minneapolis Shoal Light was built from the same plans.

buttress. From the top of the wall a 12-in. concrete deck slab extends inward to the house walls, forming the unloading and servicing deck of the pier. The massive character of the concrete and crib structure is apparent.

### Cribs towed to sites

The cribs for both lighthouses (Grays Reef and Minneapolis Shoals) were constructed in a protected harbor and towed to location, with the steel belt for ice protection in place. This belt is 10 ft. wide, of ½-in. plate. At the site the cribs were spotted and sunk to a dredge-smoothed bottom by loading the stone pockets or interior cells with conveyor stone from self-unloading boats. Divers chinked the bottoms of the outside cells, and they were filled with 1:3:5 concrete by tremie. The final operation involved the placing of a belt of riprap around the bottom of the crib.

FIG. 4—TIMBER CRIB DETAILS for Greys Reef Lighthouse, showing elaborate bolting and tierodding, to insure rigidity.


Outline of building
Sym. about C.L.
A
A
Special splice
24"x96"x24 Ga. Bl. iron sheets alternated on timbers
6" E.H. pipe
10'-2"
10'-2"
64'
8'-1½"
8'-1½"
5'-6"
5'-6"
6"x12" planks
Plan
Timber sizes sym. abt. C.L.
1"-36" thrd. rods
1"-39½" thrd. rods
12"x12"x20'
12"x12"x32'
12"x12"x6'
12"x12"x24'
Filled with concrete
Filled with concrete
10'
4'-6"
24'
64'
Special splice
6"x6" plank
Section A-A

### Steel sheetpile pier

At Peshtigo Reef, with only about 10-ft. depth of water, a considerably lighter pier structure than that previously described was adopted. Steel sheetpiles were driven in about a 50-ft.-diameter circle, to hold a stone fill within which the lighthouse structure was carried on five concrete-filled steel cylinder piers founded on clusters of six wood piles. The construction is shown by Fig. 5. Incidentally the house is designed with a minimum of living space as it is planned to be entirely unwatched, and is automatically operated with certain functions remote-controlled by radio. In general detail the structure and construction are as follows:

As shown by Fig. 5, the support for the lighthouse structure is five cylindrical piers, 6 ft. in diameter, carried by timber bearing piles, a cylinder of steel plate serving as a form for a concrete pier resting on the piles and extending up to the under side of machinery-floor level. A ring of sixteen piles, consisting of concrete-encased H-beams, was driven by jetting on a line 2 ft. 1 in. within the outer face of the sheetpiling. The ring of steel sheetpiling was then driven and stiffened at the water line by a curved cylindrical girder and secured by tierods to the inner foundation piers.

A concrete deck at the level of the machinery floor is supported on the tops of the inner piers and by brackets from the H-column piles and is extended out against the sheetpiling wall. The machinery-room story of the building proper is of concrete carried up above deck level and lighted by ports as shown. The main pier deck is constructed integrally with the walls of the machinery room and is carried at its outer edge on the steel sheetpiling ring and on the H-column piles as shown, so that the stability of the entire structure is independent of settlement in the stone fill, which is placed inside the pier, and also of deterioration that might take place in the steel sheetpile ring.

The superstructure is entirely of steel, that portion devoted to living quarters being backed up with hollow-tile masonry laid in cement mortar and well insulated. Similar wall construction is used against the sheetpiling in the storeroom at machinery-floor level. A protective mat of riprap is placed around the pier to minimize any tendency to erosion.

### Concrete-encased crib

At Green Bay light station in about 26 ft. of water, return to heavy pier construction is shown by Fig. 7. Heavy ice attack is likely at the location. The pier is about 50 ft. in diameter, consisting of a circular wood crib about 44 ft. in diameter encased in a reinforced-concrete wall from top to lake bottom, 3 ft. thick and capped by a 2½-ft.-thick concrete deck slab carrying a heavy circular concrete wall and the concrete house. Concentric rings of steel plate around the crib provide forms for the concrete encasement and also an outer steel armoring to protect the concrete from abrasion. The remaining details of structure are shown by the drawing.

### Superstructure and equipment

As they are typical of modern practice, tower structure and equipment warrant brief attention. For this purpose the Grays Reef structure is selected. As shown, the tower is a steel frame covered with steel plating carrying the light 82 ft. above water level. The equipment, which may be taken as typical of a modern station, consists of an electric light, giving about 100,000 cp. through a fourth-order fixed lens in the lantern; also a compressed-air tyfon fog signal, installed in duplicate and operated simultaneously, to cover both directions of approach, and a radio beacon. The latter signals are transmitted from a vertical antenna supported on top of the lantern above the tower. The lantern was designed especially with support of the antenna tower in view, this type of antenna having been selected because of its advantages in more uniform field pattern and reduction in night effect.

To provide power for this signaling equipment, the station is equipped with the following machinery: two 30-hp., oil-engine-driven air compressors for fog signal; two diesel-engine-driven generators of 10 hp. each, for supplying light and radio beacon with 200 amp.-hr.; a 110-volt battery, floated on the line, to provide for light loads and starting current. In addition, there are starting compressors for the fog-signal engines, air-hoist motors for the two derricks, and motor- and clock-controlled devices for timing radio beacon and other signals.

### Renovation program

The new lighthouses described are part of a new program that includes many other operations for which PWA money was granted. Normal public-works appropriations have as a rule been sufficient only for the most urgent replacements and improvements. In the summer of 1933, when an enlarged public-works program was made possible through funds allotted by PWA, the lighthouse service was able to estimate more adequately for its needs in modernizing plant and equipment, with the result that a program of work totaling about $5,600,000 was authorized for the two-year period ending on June 30, 1935.

It would be difficult to indicate in brief space the diversity in character of the nearly 200 projects involved in this program. Some idea may be obtained from the following approximate summary: ten major light stations—four to replace lightships and six on submarine sites; five new depots and extensive improvements at ten others; two new district office buildings; three new lighthouse tenders; ten new dwelling structures, including two-, three- and four-family types, and extensive repairs to many others; two derrick barges; six-

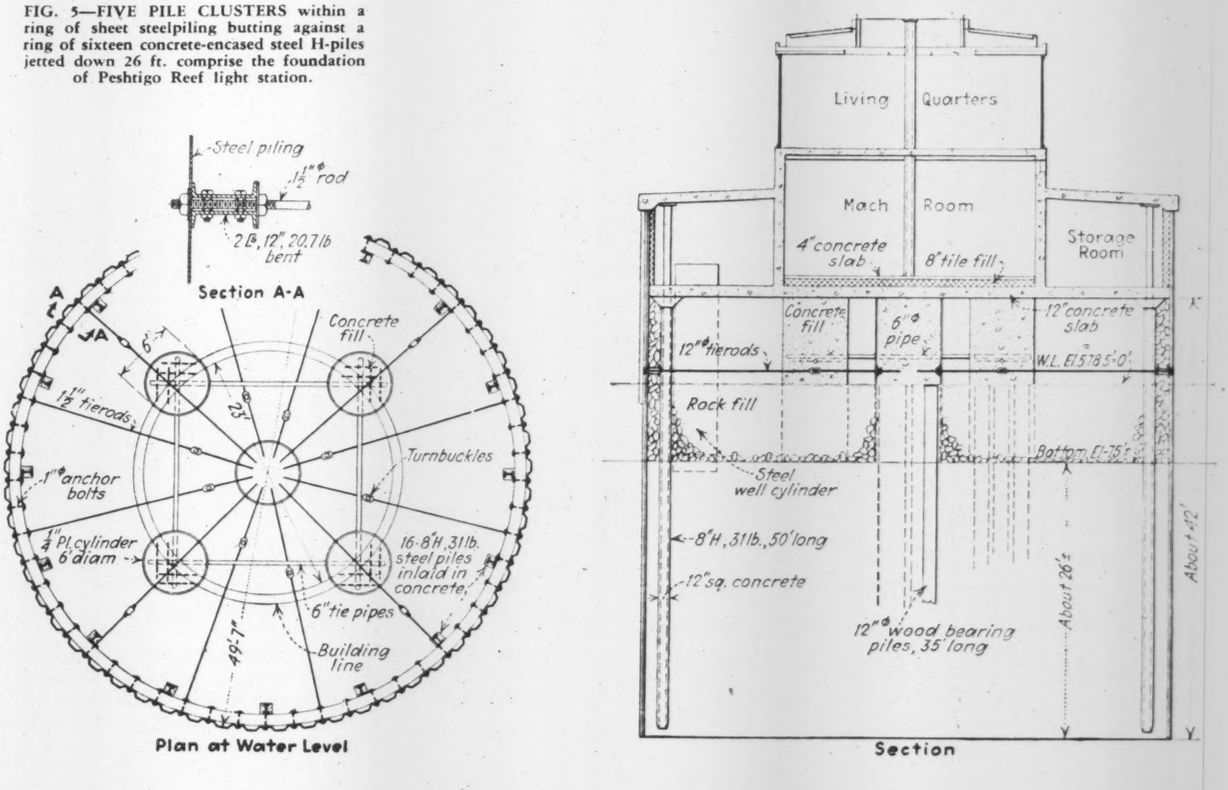

Steel piling
$1\frac{1}{2}$"ϕ rod
2 [s, 12", 20.7 lb bent
Section A-A
A
A
Concrete fill
6'
23'
$1\frac{1}{2}$" tierods
Turnbuckles
1"ϕ anchor bolts
$\frac{1}{4}$" Pl. cylinder 6' diam
16-8" H, 31 lb. steel piles inlaid in concrete
6" tie pipes
49'-7"
Building line
Plan at Water Level
Living Quarters
Mach Room
Storage Room
4" concrete slab
8" tile fill
Concrete fill
6"ϕ pipe
12" concrete slab
12"ϕ tierods
W.L. El 578.5'-0'
Rock fill
Bottom El-75'±
Steel well cylinder
8" H, 31 lb., 50' long
12" sq. concrete
12"ϕ wood bearing piles, 35' long
About 26'±
About 42'
Section

FIG. 5—FIVE PILE CLUSTERS within a ring of sheet steelpiling butting against a ring of sixteen concrete-encased steel H-piles jetted down 26 ft. comprise the foundation of Peshtigo Reef light station.

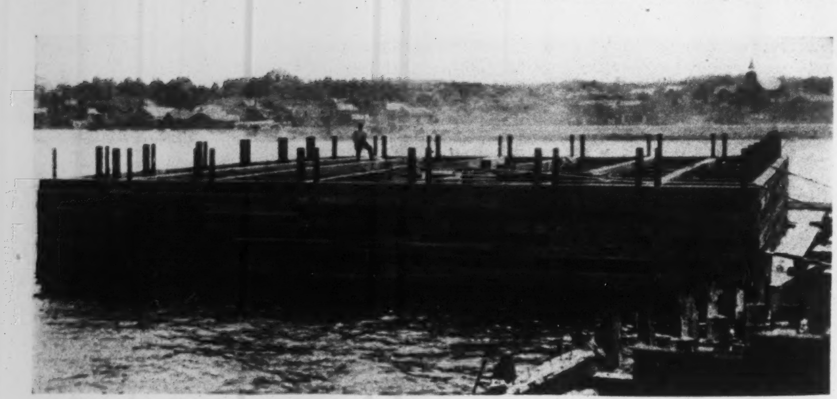


FIG. 6—FOUNDATION CRIB for Grays Reef lighthouse afloat and nearing completion before installation of ice plating. Note joining angles at outer intersections of timbers.

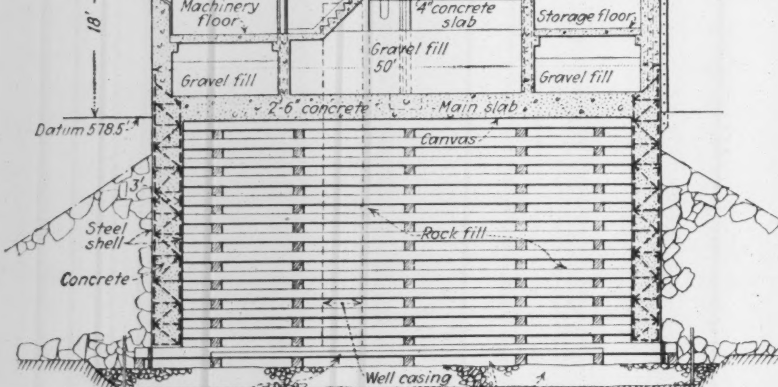
teen lighthouse tenders extensively overhauled, including reboilering, change to oil-burning, etc.; six lightships overhauled extensively, as above; thirty stations and vessels improved by the installation of new and modern radio transmitting equipment.

In addition, a great many new buoys and sets of automatic lighting equipment have been provided. Riprap protection has been provided for many pier structures, boat houses have been built, many fog signals have been improved by installation of higher power signals, and the power of many lights has been increased by electric installations, docks and wharves have been rebuilt, and shore protection is provided. Two of these renovation operations are shown by Figs. 1 and 2.

## Cost and administration

The cost of the Grays Reef Station will be about $200,000, while that of the Minneapolis Shoal Station will be about $155,000. The contractors for the Grays Reef Station are Greiling Brothers Co., Green Bay, Wis., and for the Minneapolis Shoal Station are Luedtke Engineering Co., Frankfort, Mich. Another similar station in Lake Michigan, replacing North Manitou Shoal lightship, is being built at a cost of about $150,000. Its construction is somewhat similar to those shown, but steel sheetpiling is provided to protect against erosion, and the superstructure is of steel channels, effectively assembled for mass architectural effect. Other structures at Green Bay, Wis., and Peshtigo Reef, Wis., both on Green Bay, costing $90,000 and $80,000 respectively, are being completed, the latter replacing a lightship and being designed for remote control by radio from a station on shore, 12 miles distant. A lightship in Lake St. Clair will be similarly controlled from a distant station by the use of radio, following a method that has been in practical operation for nearly three full seasons between stations 4 miles apart.

FIG. 7—GREEN BAY LIGHT STATION in Wisconsin stands in deep water, where it will be subjected to very severe ice action. It is of heavy construction with a solid exterior wall of concrete extending to bottom and a main reinforcing slab at water level.



Actual work of carrying out these projects is under the immediate supervision of the superintendent of lighthouses, within whose district the work is located. The 12th District, within which the Grays Reef Station is located, is in charge of C. H. Hubbard, senior engineer and superintendent of lighthouses, and N. M. Works, assistant superintendent and lighthouse engineer.

There are seventeen of these lighthouse districts. Detailed plans are prepared by the technical staffs of these offices and by the engineering division of the Bureau of Lighthouses at Washington, D. C. In the case of the Grays Reef and Minneapolis Shoal structures and some of the others herein mentioned the Commission of Fine Arts cooperated in reviewing the plans from the standpoint of architectural treatment. The Bureau of Lighthouses is administered by the commissioner of lighthouses, George R. Putnam, and the deputy commissioner, Harold D. King. The engineering division includes Charles A. Park, chief engineer, E. C. Gillette, marine engineer, F. C. Hingsburg, signal engineer, and G. B. Skinner, architectural and structural engineer.

◆

## Survey for Artesian Water Under Way in Canada

In order to alleviate the drought conditions in western Canada, the Geological Survey is conducting water investigations in southern Saskatchewan and southwestern Alberta.

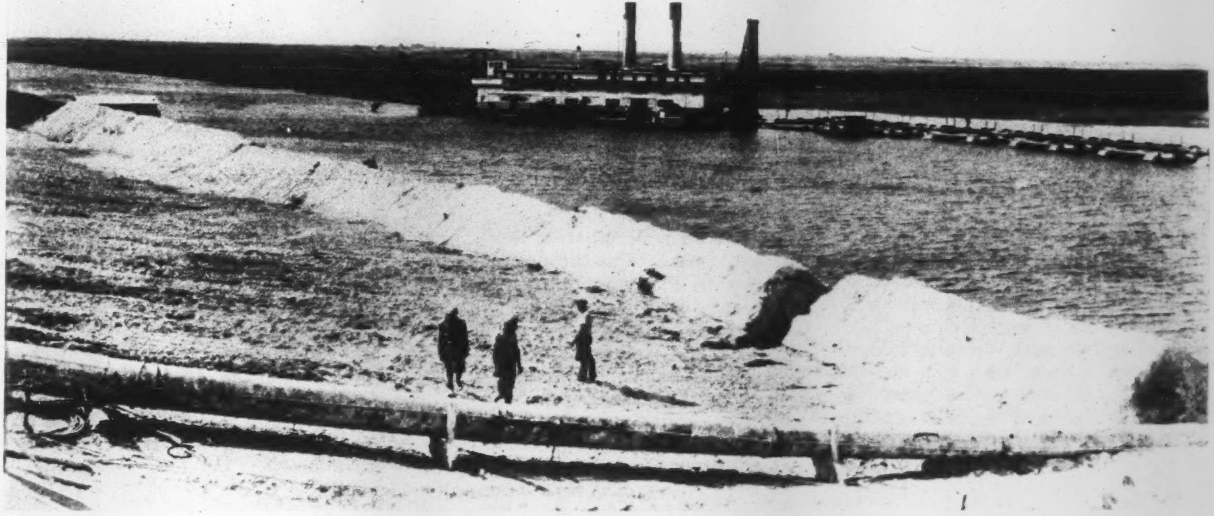
Since the discovery of the Milk River artesian well area in southern Alberta, the Survey has been asked for information regarding the depth of the water table and the quality of water to be obtained. The work is being done as one of the projects in the $1,000,000 program of the Geological Survey provided for in the Canadian public-works construction act of 1935.

Parties under competent geologists will examine existing wells, make borings, and collect data from which to deduce the source and quality of the water to be obtained by boring.

# Cheap Levees by Hydraulic Fill at Lake Okeechobee

**Hydraulic dredges, with special-type cutters, are handling a mixture of sand, marl and blasted rock at strikingly low cost in constructing 66-mile defense against storm-wave inundation**

Hydraulic dredging methods are being employed at considerable savings in cost over bucket-type excavators in placing much of the 50,000,000 cu. yd. of materials required to construct 66½ miles of levees along the south shore of Lake Okeechobee at the head of the Everglades in Florida. These methods of handling material also produce a better embankment for the conditions involved than was obtained earlier on this project with bucket machines. The results are specially significant in view of the fact that the materials encountered range all the way from soft sea sand and marl to layers of hard limestone.



HYDRAULIC DREDGE pumping into flood-protection levee at Lake Okeechobee. Dikes to provide a pond for hydraulic fill are constructed well in advance of pumping.

The general features of the Lake Okeechobee flood-protection and navigation plan were described in *Engineering News-Record*, Aug. 16, 1934, p. 195. In that article the conditions met in designing and building the levees along the shores of this shallow lake were summarized.

Under the plan the St. Lucie Canal, leading directly east to the Atlantic, is being improved to provide a normal carrying capacity of 5,000 sec.-ft. Also Caloosahatchie River, extending from the west shore of the lake to the Gulf of Mexico, is to be dredged and widened to provide an outlet in that direction with a capacity of 2,500 sec.-ft. As a part of the plan an 8-ft. channel is also being provided across the lake. This channel is for the most part 200 ft. offshore from the levees, but in some locations, varying with the materials encountered and the type of levee constructed, the channel is as much as 1,000 ft. offshore. Its eastern end connects through the St. Lucie Canal with the Atlantic, and to the west the improved Caloosahatchie River channel will provide an outlet to the Gulf. There will thus be available an 8-ft. channel entirely across the Florida peninsula, which will carry a large volume of pleasure-craft traffic and may eventually handle considerable commercial business.

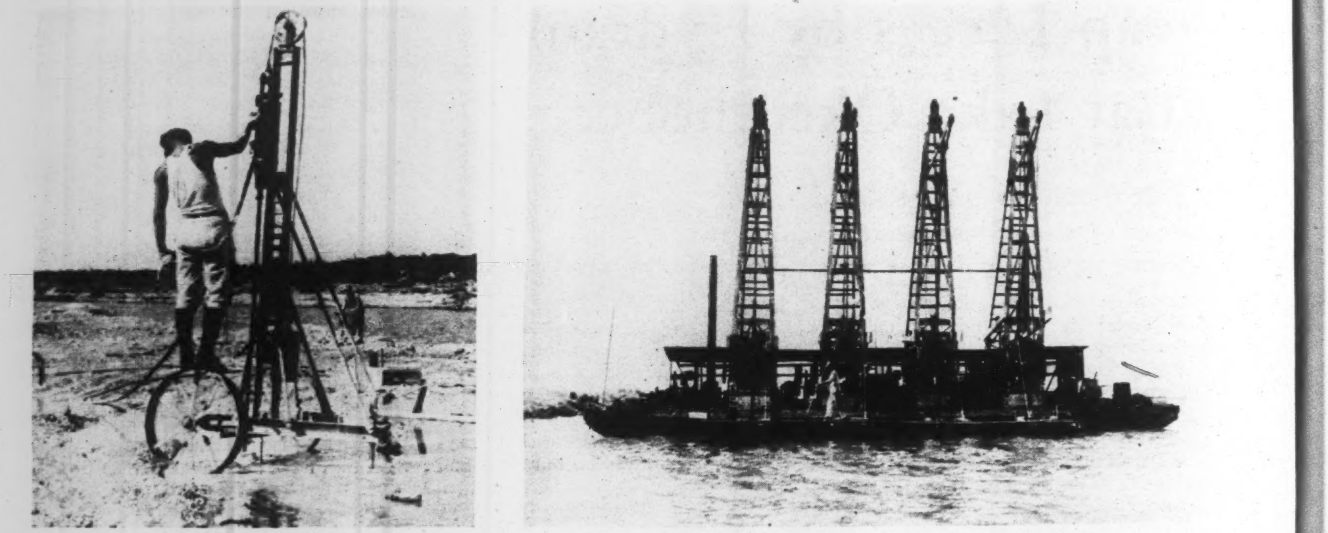
### Material and water conditions

The bed of Lake Okeechobee and the surrounding areas are still in the process of formation. Materials vary widely in short distances, with uniform deposits the exception. There usually are, however, underlying strata of marl or limestone. In many places these materials come close to the surface; in others they are overlaid to considerable depths by seashells, sand and material that is almost colloidal. These overlying materials also generally contain strata of marl and limestone varying from a few inches to several feet in thickness. As a general rule, the surface is a layer of partly decayed growths of reeds and swamp grasses, woody growth being quite rare.

Water depths are slight along most of the levee locations. In fact, the lake itself is in no place more than 20 ft. deep, with less than that much water over practically all of it. There is ample water, however, to permit operation of floating equipment, and by borrowing material for the levees from the 8-ft. permanent channel much greater depths than that have been available at times.

From the beginning the engineers specified that the embankments might be built either by dragline excavator or by hydraulic dredge. The sections for the two methods of placing materials varied markedly, with much flatter slopes and about 15 per cent more material in the hydraulic-dredge type. With one exception, all of the qualified low bidders on the first 37 miles of levee for which contracts were awarded elected to use dragline excavators.

As a general proposition, good results were obtained with the dragline machines. A section begun with hydraulic dredges by one contractor, though inadequately equipped, nevertheless proved that hydraulic dredging offered opportunities for constructing levees under the local conditions more cheaply than dragline excavation. The resulting embankment also was more stable. Based on these conclusions, the engineers elected to take over a section of levee on which the contractor was badly be-



TYPES of drilling outfits used in borrow-pits, which furnish rock mixed with marl and sand to be placed in the levee by the hydraulic dredges.

hind schedule, in order to give the hydraulic-dredge method a thorough trial. The engineer department's 20-in. dredge "Welatka" was brought through the St. Lucie Canal and across the lake to the site.

Out of the experience with this dredge it became evident that exceptional power was required on the dredge cutter shaft. Cutters specially designed for the local conditions also were found necessary. The angle and depth at which the cutter worked had a great effect on the amount of material delivered. Various other fundamental conditions were determined. But the advantage of a properly equipped and properly powered hydraulic dredge over any kind of a dragline machine on this particular work was evident. Some hard material encountered by the dredge on this first section had been handled with little difficulty, in spite of the lack of sufficient power on the cutter shaft. There was still serious doubt, however, among the hydraulic-dredge advocates whether this method could be used successfully in sections where considerable rock was encountered. Contractors on such sections, who had drilled and blasted ahead of their dragline machines, were certain that no hydraulic dredge could meet the conditions.

## Development of dredge methods

Opportunity to give the hydraulic-dredge method a trial under the most difficult material conditions occurred when a contractor using draglines failed after placing a considerable part of the base of a section of levee. The engineers leased from the Arundel Corp. the 24-in. heavy-duty dredge "General," and delivered material from the borrow area adjacent to the offshore toe of the levee to the end of the discharge pipe along the embankment for approximately 9c. per cu. yd.

This dredge is modernly equipped, has ample power, is fitted with a heavy cutter ladder and cutter shaft; has 500 hp. on the cutter and 1,800 hp. on the pump, the cutter unit having a 25 per cent overload capacity for short periods. This dredge placed 1,060,000 cu. yd. of net material in the levee in one 90-day period, and the best month's output was 395,000 cu. yd. On these particular sections loss of material from the borrow-pit measurements to the pay quantities in the embankment averaged 20 per cent.

Where the hydraulic dredge places the entire levee, the first operation is to throw up low retaining dikes along both toes with small dragline machines. These usually are kept at least half a mile ahead of the point where the dredge is discharging. They are built high enough to retain the main fill to its entire height on the final slopes. The discharge pipe is then erected on temporary bents along the axis of the main embankment. With stop boards it is easy for an experienced crew to direct the material delivered by the discharge pipe of the dredge almost exactly to the designed slopes. More or less of the very fine material continues in suspension so that it is carried ahead of the discharge point for varying distances before settling.

In some localities the percentage of colloidal material is large enough to present a problem. Moving ahead in considerable amounts, it drops to form the base of the levee to carrying depths, and this base layer of fine material drains very slowly. Erosion of this material when exposed in the face of the embankment where wave action may reach it goes on very rapidly. Such conditions have occurred only occasionally. In most instances the bulk of the finer material settles rather promptly after being discharged, the colloidal matter being carried away in the waste water.

Where the hydraulic dredge is placing the entire embankment the practice has been to locate a temporary wood spillway in the retaining dike along the offshore toe of the levee as an outlet for the water from the dredge discharge pipe. The location of this spillway depends on the materials and working conditions. The best arrangement is to have it so placed as to obtain the maximum possible settlement of the desirable materials between the retaining dikes.

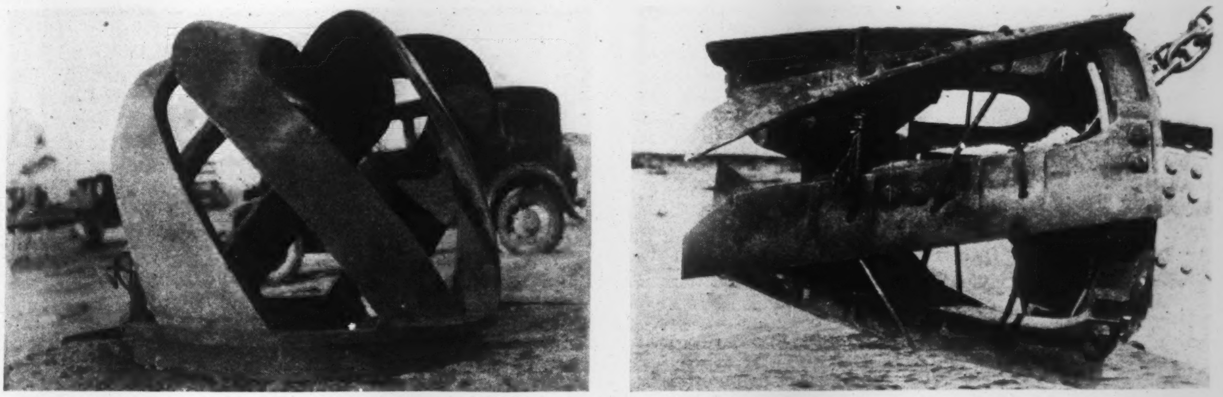
These more or less general standard methods have been followed in operating the dredge "Welatka" and the engineer department's dredge "Gulfport," which are both in service on the project. In the case of the dredge "General" the discharge pipe is placed slightly inshore from the axis of the levee so as to place the materials as nearly as possible in final location. As the base of the embankment that this dredge completed was thrown up by dragline, some rehandling of the material delivered by the dredge is necessary to secure the final slopes desired. Draglines do the bulk of this rehandling, and blade road machines drawn by tractors do the fine finishing. Even with this combination dredge and dragline operation the final cost of material in place, including grassed slope protection, does not exceed 14c. per cu. yd.

On 5.5 miles of levee undertaken under contract by the Arundel Corp. the latter started operations with large land dragline machines as the main units of its plant, supplemented by smaller draglines for various auxiliary operations. Later this contractor developed a combination dragline and hydraulic-dredge method of placing the embankment. As a hydraulic dredge immediately available lacked sufficient power to operate to the best advantage, a plan of placing materials was adopted that gave the best results under the circumstances.

Low retaining dikes were thrown up ahead in the usual manner. Then the dredge filled about to the top of these dikes without the use of stop boards, leaving a virtually flat surface. Next, the large dragline rehandled part of the hydraulically placed material to bring the retaining dikes up for another lift to be placed by the dredge. In this sequence the embankment was carried to the proper height, and with standard dragline section slopes. By this method, therefore, materials handling was reduced.

Based on the earlier experiences with the "Welatka," the original ladder on this dredge was replaced with a much heavier one. The original 60-hp. engine for handling the ladder and swinging lines was replaced with a 140-hp. motor. A 250-hp. steam engine on the cutter was replaced by a 490-hp. motor able to handle 10 per cent overload for some time.

Thus re-equipped, the "Welatka," has shown the value of ample power for the heavy rock excavation, although at the time these notes were prepared she had been in service less than 30 days. During that short period the dredge handled as high as 12,000 cu. yd. of material delivered in place in the embankment in 24 hours.



TYPES of dredge cutter heads used. The one at the left is for soft material, and that at the right is for blasted rock. In the rock type, wire rope threaded through the cutter blades screen out the oversize rock.

## Types of cutters

Basket and open-blade cutters have been used on the three large dredges on this project. The basket type, suitable only for soft materials, is used only to limited extent. On account of the very heavy service in hard material and rock, modifications of the usual open-blade type of cutter have been found desirable. Accompanying views of these cutters show the spacing and length of blades and other features. In these views attention is called to the use of wire cable threaded between the cutter blades. This is done to keep larger pieces of rock from entering the suction than will readily pass through the pump and the discharge line.

Due to the constant variation in the materials as they occur in the borrow-pit, no fixed method of dredge operation is practicable. This variation also affects seriously the amount of material delivered from time to time by the dredge. The angle of the dredge ladder and the slope of the blades on the cutter determine the output in different materials and at varying depths. In broken rock the output of fine material will run low. There thus is no way to estimate in advance the probable day-to-day output of a dredge on the project. There appear to be possibilities of increasing capacity considerably by using a dredge equipped with a ladder having one angle and a cutter with one pitch of blades to a certain depth, and another dredge following to finish the excavation with another angle on its ladder and a different cutter pitch, but this has never been tried.

Handling of the thin strata of hard materials ranging from marl to hard limestone is one of the outstanding features of the hydraulic-dredge operations. In all cases where the contractors have placed the embankments with dragline machines these strata have been drilled at 8-ft. spacing in both directions. When excavating under water, floating drill boats have been employed. Where the dry-pit method of excavating was used, holes were made with portable land drills. Loading has been done with 4 to 6 lb. of 60 per cent dynamite to the hole. On account of the thinness of the hard strata, varying results have been obtained in blasting. There has been some question as to whether blasting has in all cases been effective enough to justify it. Blasting in connection with the hydraulic-dredge operations has followed about the same procedure as in dragline work.

The three large hydraulic dredges have handled the broken rock with surprising effectiveness. In fact, this has been an outstanding feature of the operation of these dredges on the project. Local opinion on the project inclines to the possibility of eliminating blasting of the hard strata by using the dredges to undermine them and cause them to drop below the minimum channel depth or be broken by the cutters to such extent that the bulk of them may be dredged.

## Experience with retaining dikes

One problem in building the embankments entirely with hydraulic dredges has been the lateral movement of the low retaining dikes along the toes of the slopes. These dikes frequently for some distance are on the muck that forms much of the bed of the lake. This muck compresses up to 35 per cent under the maximum depth of levee embankment. It is fairly watertight when forming the side of a pit, but under some conditions it moves easily on the underlying materials.

Generally the hydraulic fill brings little thrust against these retaining dikes. When it does, side slips occur over limited areas that cause some loss of material delivered by the dredges. Volumes of pumped material thus lost average much less than the yardages that have to be rehandled due to slips on much of the work done with draglines.

## Protection against wave action

Protection against wave action on the lake side of the levees is also a problem that is still being studied for the best solution. This is not so serious on most of the levee placed with hydraulic dredges, but is very important on the dragline sections. With an exposure of 30 to 35 miles of open water for long stretches of the levee, fairly heavy wave action may occur. Probably more serious as traffic develops, will be the wave action from fast motor craft in the adjacent channel. Protection from tidal waves and rainfall appear to be taken care of by heavy grass growths on the slopes.

Trials were made of concrete slabs extending from about 1 ft. below to 4 ft. above normal low water, which is well above any lake stages except during hurricanes. As the lake will occasionally fall or be drawn below the normal low water, the slabs would be exposed to undermining. To build them deeper than 1 ft. below normal low water would entail too large an outlay. Hence this type of protection was abandoned.

Rock riprap, 2 ft. thick and extending from 2 ft. below to 4.5 ft. above normal low water, has been placed along the offshore toe of about 18 miles of the dragline-section levees. This rock ranges from one-man to 1,000-lb. pieces. Most of it was obtained from a channel excavated as a floodway in another part of the lake. One contractor salvaged pieces of rock from his dragline excavation for this purpose.

Riprap protection is satisfactory, as erosion of material below it merely causes settlement of the rock to a new bed. The cost of this type of protection under the local conditions is relatively high, however. Other types of toe protection accordingly have been studied extensively. An experiment has been undertaken to determine whether one of the heavily-rooted coarse reed grasses of the locality will grow profusely enough when protected during its early stages, to provide permanent embankment toe protection. Temporary bulkheads of waste lumber were built along the toe of a levee slope for several hundred feet at nominal cost as an experimental protection. Stools of grass have been planted along the water line behind these bulkheads. The expectation is that enough growth will occur before the bulkheads decay or are washed out, barring a hurricane tidal wave meantime, to protect the toe against the wave action to which it may be subjected.

Consideration has been given to the use of beaches of shell pumped on the flat lakeside slopes of the hydraulic-fill embankments where the type of material in the latter renders them liable to wash. In most cases, the 20 to 1 slope on a 30-ft. berm is expected to eliminate loss of enough material to be important.

### Direction of work

Plans for the Lake Okeechobee flood-control works were prepared by the Corps of Engineers, United States Army, and are being executed under the direct supervision of that organization. Gen. Edward M. Markham is chief of engineers, and Lieut.-Col. B. C. Dunn is engineer of the district under which the project is conducted, with headquarters at Jacksonville. First Lieut. P. A. Feringa is military assistant in that office. Carawan Nelson is engineer in general charge of the project, with Raymond C. Baird, C. C. Schrontz and Perry M. Teeple as associate engineers. First Lieut. R. Selee is resident engineer on the project, having associated with him Lieut. W. H. Mills and E. W. Digges as civilian engineer.

# Civil Engineers' Meeting Covers Western Problems—II

## Second half of program of Los Angeles convention includes papers on flume design, aqueduct construction and state and municipal services

THE PRINCIPAL SUBJECTS discussed at the 65th annual convention of the American Society of Civil Engineers in Los Angeles, July 3-7, concerned engineering problems common to the western states and particularly to southern California. The first half of the report of the meeting (published last week, p. 59) having covered all professional developments and the papers on rivers, waterways and traffic regulation, there is left for this concluding chapter consideration of papers on flume design, aqueduct construction, state planning and water supply and sewage.

### Flume design

Recent trends in the design of flumes, especially with respect to the effect of curvature and surface conditions on flow, formed the subject of an illustrated paper by Fred C. Scobey (Berkeley). Curvature in channels, Mr. Scobey stated, has not as important a bearing upon capacity as was once believed. Relatively sharp curvature can be introduced without serious losses. On the other hand, a slight increase in the smoothness of the channel interior has a marked effect in increasing channel capacity. On the Owens Valley aqueduct the introduction of a new and smoother lining, though reducing the cross-section area, increased the capacity from 400 to 500 sec.-ft. Mr. Scobey stated that values of *n* in Kutter's and Manning's formulas now used with the better types of lining that are available are about the same as those used years ago. This is because the earlier values were optimistic and did not represent what could actually be obtained.

Improvements in methods of air application of concrete in lining canals were illustrated by Mr. Scobey. Quite uniform cross-sections can be obtained by striking off the surface immediately after the concrete is shot and by light, quick troweling. If this finishing work is not overdone there is little loss in density.

In "Selection of Material for Rolled Earthfill Dams," Chas. H. Lee (San Francisco) pointed out that while the selection of material for such structures has been developed to a point similar to that now in vogue in the proportioning of concrete materials, economy usually dictates the use of local materials. The problem thus becomes one of finding the best use for the materials available. In general, earth passing a No. 10 sieve represents the mortar, and the larger material represents the aggregate of earthfill mixtures.

Several grading curves of typical materials were compared with Talbot's equation and with standards recommended by the U. S. Bureau of Reclamation. Materials having a value in Talbot's equation of 0.33 to 0.5 were found to give satisfactory stability. The compaction methods developed by R. R. Proctor (*ENR*, Aug. 31, Sept. 7, 21 and 28, 1933) were said to improve cohesion by reducing the thickness of the moisture films, and to reduce leakage. Best compaction is obtained with materials having a loam texture, as this is better than either very fine or very coarse material. Loams give maximum weight and minimum voids through which moisture may later percolate. Mr. Lee's main thesis was the use of grading curves based on mechanical analysis.

Discussion by H. A. Van Norman (Los Angeles) commended Mr. Lee's paper but expressed preference for the compaction and permeability tests developed by Mr. Proctor which, he thought, gave a better representation of elasticity. Mechanical analysis, he pointed out, does not give the proper moisture content, and the equipment and personnel essential to adequate mechanical analyses present difficulties. No final selection should be based on any laboratory tests; field tests always should be used as a check.

Further discussion by Fred Pyle and Paul Beerman (San Diego) pointed out that early earth dams used heavy sections; then came a period of steeper slopes, while still more recent construction indicates that we now are getting back to more liberal sections. Preference was expressed for other tests to supplement mechanical analysis. Results on El Capitan Dam and other structures were cited to show that a wide variety of materials will give satisfactory performance. A plea was made for obtaining complete records of settlement on dams built by different methods as well as for standardization of terms so as to remedy the wide divergence in the meanings of "sand" and "gravel" when used (1) in describing mechanical analysis of earth and (2) with reference to concrete materials.

### Aqueduct construction

At the construction division three papers were heard. Features of the design and construction of Morris Dam, Pasadena's water storage dam in San Gabriel Canyon were presented from different angles by Verne L. Peugh (Pasadena), Walter L. Huber (San Francisco), and Kenneth Q. Volk (Los Angeles). Tunnel driving methods and progress on the Colorado River aqueduct were described by J. L. Burkholder (Los Angeles), who began with a sound movie which seemed actually to take the audience down the shafts and into the tunnels. Excellent action photography, the sound of blasts, drills and power shovels in operation together with the accompanying descriptive comment made an impressive presentation of what men and machines are doing on this job. Sixty per cent of the tunnel driving is finished and nine miles of tunnel have been lined, he said, on this $209,000,000 project, which is scheduled for completion in December, 1938. The job now is practically half district work and half under contract. Thirteen contracting firms have more than $10,000,000 invested in equipment on contracts totaling $85,000,000. Excellent progress is being made; on one of the 10-ft. tunnels of the distributing system (of which the total length will be 150 miles) a recent 30-day period showed an advance of 1,795 ft., and as much as 85 ft. has been made in a single day's work.

The third paper of this session, "Large Siphons Under High Heads," by N. D. Whitman (Los Angeles), gave particular attention to design and construction of precast concrete pipe in sizes up to 12 ft. in diameter whose 12-ft. lengths weighed 40 tons. Discussion by D. B. Gumensky, Los Angeles, outlined principles and objectives which governed design under the varying conditions met on this job.

### State planning

The paper on "Objectives and Progress in State Planning" by L. Deming Tilton (Los Angeles) set up as basic objectives of state planning board:

(1) Provision for a greater measure of health, safety, comfort and security for every citizen.

(2) Opening of new avenues for enrichment of life.

(3) Conserving and developing with maximum efficiency all human physical resources.

(4) Providing social equipment which will more adequately reflect the natural wealth and advantages of the state.

Heretofore the growth of the state has been haphazard, with much waste of natural resources. This has been costly; the planning board should serve as a clearing house for correction of such conditions. Time will be required and handicaps confront the plan, but there are great advantages to be gained if data, reports and recommendations can be presented effectively.

Gordon Whitnall (Los Angeles), pointed out that the work of a state planning board is pioneering, is broad in its scope and requires specialized knowledge of existing governmental agencies. Harry Bergh, of the Los Angeles County Regional Planning Commission, described the set-up under which that body operates. Lack of authority to enforce its recommendations is often an advantage, he said, rather than a handicap because this makes it necessary to secure the support of public opinion to back up unbiased recommendations. R. R. Lyman (Salt Lake), stressed the need for having the engineer and the social worker get together for the promotion of human welfare.

### PWA experiences

A paper on "The Use of Plans and Planning in Programming Public Works" was presented by Wm. J. Fox (Los Angeles). A. D. Wilder, PWA state engineer for California, explained how he had quickly built up an organization for handling PWA work, how each project was scrutinized before loans or grants were made, how assurance was obtained that the loans were properly secured, and how all this was done with efficiency and dispatch under a constant demand for speed. J. C. Albers, city enginer of Glendale, said a severe flood in January, 1934, had shifted the needs in that city to emergency repair work on stream channels, bridges, etc., and that this work was speedily organized and carried out with financial aid from the government.

Arthur S. Tuttle, PWA state engineer for New York, endorsed the view that work projects for the unemployed are vital emergencies. He spoke of the tentative program of projects which had been hurriedly presented to the New York state planning board and how the board had to consider and speedily select the more worthy ones. O. E. Carr, PWA state engineer inspector for California, said that some $900,000,000 of the $3,300,000,000 public works appropriation had gone to nonfederal projects and of this amount some $350,000,000 had gone to the railroads, leaving only about $550,000,000 for projects in which federal and state planning would be required. A large part of this last figure is allotted to schools, he said.

### Chlorination at Los Angeles

Papers presented before the sanitary engineering division were concerned with water problems at Los Angeles and with sewage disposal matters at Pasadena. At Los Angeles, according to Ray L. Derby, the outstanding improvement in chlorination has been the installation of distribution headers for the application of chlorine solution into lines to be treated. In three installations with pipe sizes 36, 45, 54, and 72 in. in diameter, respectively, single points of application caused erratic mixing, such that residual chlorine in the treated water at various points in the cross-section of the pipe lines at distances 250 to 400 ft. below the chlorinators varied from 0 to 3.5 lb. per million gallons. Through the installation of headers whereby the chlorine solution is fed in at nine points instead of one the residual chlorine at all points in the cross section has become uniform, effecting a 50 per cent reduction in chlorine doses.

All Los Angeles chlorinators use injector pumps for chlorine solution lines even where gravity lines are treated, in order to increase the chlorine solubility in the solution water and keep size of solution lines to a minimum. Other improvements include correlation between turbidity and residual chlorine requirement, and the making of standardized ortho-tolidin readings.

### Pasadena sewage disposal

The Pasadena activated sludge plant, which was the subject of a paper by Harvey W. Hinks (Pasadena), treats sewage from 115,000 people and has a per capita flow of 63.5 gal. per capita per day. Nitroganic fertilizer is produced which has less than 30 bacteria per gram as compared with 8.5 million bacteria per gram in the undried sludge. Portable elevators for stacking and loading fertilizer in 100-lb. paper sacks have proved to be handy. It is estimated that the city saves $30,000 a year in drying the sludge for fertilizer as compared to the previous method of burying the partially dewatered filter cake. The nitroganic fertilizer averages 6.53 per cent ammonia, which is higher than what was anticipated.

A paper on "Odor Control at the Pasadena Activated Sludge Plant," by O. H. Hedrich (Pasadena), reported on the research and experimentation that has been carried on by the Pasadena plant in trying to control odors from the sludge dryer. Experiments using the following ideas proved on the whole unsuccessful: removal of odor by water sprays, filtering of exhaust dryer gases through charcoal and chlorine gas, cooling of exhaust gases to condense odor-containing substances, addition of ozone with or without chlorine, exhausting gas into raw sewage and into final effluent, and addition of "Odor-Nox."

Moderate success has been obtained by incinerating exhaust gases at 1,800° F. Vertical furnaces are preferred over horizontal in that they take less ground, require no track, and give better air inlet control. Odor control by use of high temperature is not perfect, and costs are relatively high. At the present time the city is experimenting with the use of heat exchangers operating on exhaust gases which have been cleaned in spray towers. This plan is expected to reduce the fuel cost. The second line of investigation is in bubbling exhaust gases through sodium hydroxide, which appears to have considerable merit.

# Cellular Steel Breakwater for Calumet Harbor

**Structural design—Failure of isolated cells by Lake Michigan wave action—Recalculation of design strength—Similar lake structures compared—Method of construction—Quantities and costs**

By T. L. Condron
*Condron & Post, Consulting Engineers, Chicago, Ill.*

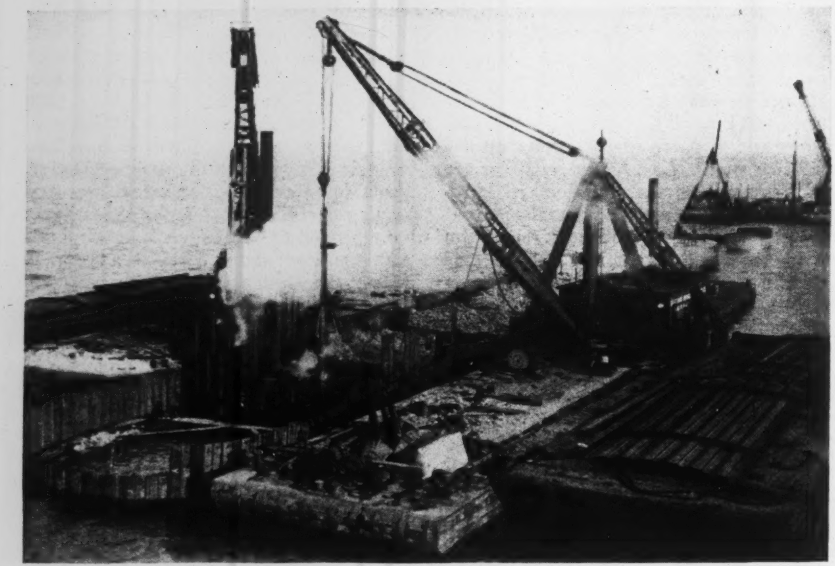


FIG. 1—BREAKWATER CONSTRUCTION of steel sheetpile cells filled and capped with stone at Calumet Harbor on Lake Michigan.

CALUMET HARBOR is located near the extreme southern end of Lake Michigan at the mouth of the Calumet River, where the Indiana-Illinois state line meets the lake. Prior to 1905, the U. S. government built a breakwater (Fig. 2), extending out into the lake practically due east for 4,200 ft. and thence southeast for 2,400 ft. The outer portion of this breakwater is a rockfilled timber crib (Fig. 3). In 1933 an extension of this breakwater, as shown by Fig. 2, was designed by the U. S. Engineer Office, Chicago, the construction to be financed with NIRA funds.

Plans and specifications were issued Sept. 5, 1933, and bids were invited on three alternate designs: (1) a rockfilled steel-pile breakwater of two designs; (2) a rubble mound breakwater; and (3) a reinforced-concrete caisson breakwater. The contract was let Oct. 21, 1933, to the Great Lakes Dredge & Dock Co., for the entire breakwater, to be built on the design of Fig. 4.

The structure was to consist of 131 cells, the end cells to be circular with a radius of 24 ft. 1 in., and the intermediate cells to be circular-arc cells with an average length of 38¼ ft. and a width of 41 ft. 1 in. Special Y-piles (Fig. 4) were designed for the connections of the cell walls.

The specifications under which the steel sheetpiling was bought called for a standard make. The interlock joint was to develop a strength in direct tension of not less than 12,000 lb. per inch of length; the weight per square foot of wall was to be not more than 23 lb. The special Y-piles were to be fabricated from the type of piling proposed to be furnished, using tieplates and angles of not less than ⅜-in. thickness. It was required that the Y-pile should be capable of transmitting a direct load on any arm of not less than 10,000 lb. per inch of length of interlock. The total estimated tonnage was 7,342 tons of sheetpiles and 468 tons of special Y-piles, a total of 7,810 tons.

*Driving Sheetpiles*—The plans called for sheetpiles in the side walls 46 ft. long, to be driven to a depth of 40 ft. below datum, with their top edges at an elevation of 6 ft. above datum. The depth specified was estimated to give a minimum penetration of 6 ft. in hard clay. If the depth to hard clay was found to be greater than 34 ft. below datum, correspondingly longer piles were to be furnished by the government, and the contractor was to be paid 25c. per foot for driving the additional length. The contractor was also to be paid 50c. for each pile cut off by direction of the contracting officer.

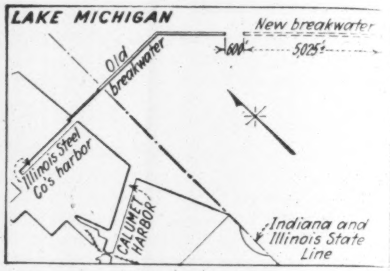


FIG. 2—CALUMET HARBOR on Lake Michigan, showing old and new breakwater locations.

The specification required that the driving of a cell should be continuous, the piles to be driven to templets furnished by the contractor. Should it not be possible at any time to drive the piles to the lines or arcs specified, the variation was to be such as to decrease the arc. Tendency to lose distance at the top or bottom of a pile was to be corrected by the use of a taper pile.

*Stone Filling*—The specifications provided that the stone filling be deposited in the cells so as to produce nearly uniform deflections in all sides. The filling stone was to weigh solid not less than 160 lb. per cu.ft., and in mixture not less than 90 nor more than 100 lb. per cu.ft.

Above the filling stone a layer of bedding stone was specified, approximately 3 ft. thick, to serve as a foundation for the capstone. The capstones were to be fairly uniform in thickness and rectangular in cross-section and were to be placed with close joints and to form a regular and uniform top surface at an elevation 6 in. above the top of the side walls of the structure.

*Riprap Protection*—Riprap protection was specified to be placed as soon as practicable after each cell had been filled with stone. After placing the riprap, a layer of toe-protection stone was to be deposited on the slope of the riprap. The riprap stone was to vary from 1 ton to 20 tons each and average between 2 and 4 tons. The toe protecting stone was to be a well-graded mixture ranging in weight from 10 to 150 lb. each.

### Delay and storm damage

Actual construction began with the setting of cell No. 66 at the mid-point of the proposed length, on May 28, 1934. The steel sheetpiles were framed into curved panels, each of thirteen piles for the side walls and of nine piles for the cross-walls, held by curved channels and angles temporarily bolted to the piles. These panels, together

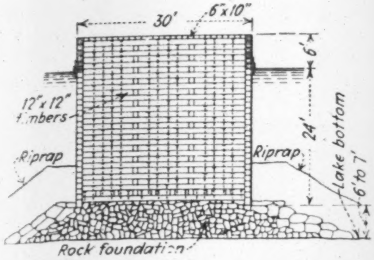

30'
6"x10"
6'
12"x12" timbers
Riprap
Riprap
24'
Lake bottom
6' to 7'
Rock foundation

**FIG. 3—OLD timber-crib breakwater at Calumet harbor that withstood the storm that wrecked isolated cells of new breakwater as it was under construction.**

with the special Y-piles and the loose closing piles were driven by noon, May 29, and filling stone from scows was placed inside the cell by derrick until it appeared above the water surface for a width of 8 ft. for the entire perimeter of the cell at 6 a.m. May 30. At that time 2,032 tons, or 82 per cent, of the filling stone was in the cell. Work was then begun on the next adjoining cell, No. 67, at noon of May 31, and driving was completed by 5:30 a.m. June 1. Stone filling was started at 2:30 p.m. that day and continued until 5:45 a.m. June 2, when a total of 1,606 tons, or 71 per cent, of the filling stone was in place, the apex of the stone pile appearing above the surface of the water.

Work had also been started on cell No. 68 on June 1, and by 2:30 p.m. four panels of sheetpiling had been set ready for driving when news was received that a tugmen's strike would be called at 7 a.m. June 2. The four panels were removed and placed on a scow. The filling of cell 67 was continued, until 5:45 a.m. June 2, when all of the contractors' plant was towed ashore before the strike started.

On June 6, 1934, at 7 a.m. the wind suddenly began to blow about 30 miles per hour from the northeast, causing a heavy sea to break over the partly filled cells. Up to 11 a.m. no damage was noted, although the sheeting was weaving. By 3:30 p.m. the waves were breaking over the high sheeting of the side walls and rushing through the cells from north to south, and at that hour the lighthouse keepers reported that the cells began to fail. By 8 a.m., June 7 both cells were partly destroyed.

The actual condition of the piling of the two cells as surveyed on June 8 by the U. S. assistant engineer, George T. Stewart, is shown by Fig. 5. His report on June 26, 1934, states: "Five of the (six) Y-plates are standing, with the adjoining piles still interlocked, the one exception being where the whole east arc of cell No. 67 tore loose." From 13 to 20 ft. of the upper lengths of the sheetpiling was unsupported by any rock filling, while below this the joints were all in tension due to the pressure of the rockfilling, and the lower ends of the piles were also securely held by their penetration of 6 ft. into hard clay. Mr. Stewart reported, "The connecting wall or diaphragm between the two cells remained intact, holding the Y-pile on the west entirely plumb and supporting the Y-pile on the east. Rock supported both sides of this wall nearly to the water line. The entire west wall of both cells is still standing intact, although the north and south ends are beginning to bend to the west."

The conclusions of the resident engineer, Mr. Stewart, were: "(*a*) The piles failed in the arcs and not in the Y's, which were considered the weakest part of the piling. (*b*) The west wall is entirely in tension, and except at the unsupported north and south ends shows no sign of weakness. The center diaphragm with rock on each side shows no movement whatever. (*c*) The piling was broken off in the east wall by the continual weakening, caused by the waves, at 2 or 3 ft. below the point where the rock lay against it."

After this failure, tests were made by R. W. Hunt Co. of sections of the steel piling taken from the field. The results of these tests showed the following:

| | |
|---|---|
| Ultimate strength of interlock per lin.-in., lb | 17,917 |
| Ultimate strength of steel, lb. tension per sq. in | 81,390 |
| Percentage of copper | 0.265 |

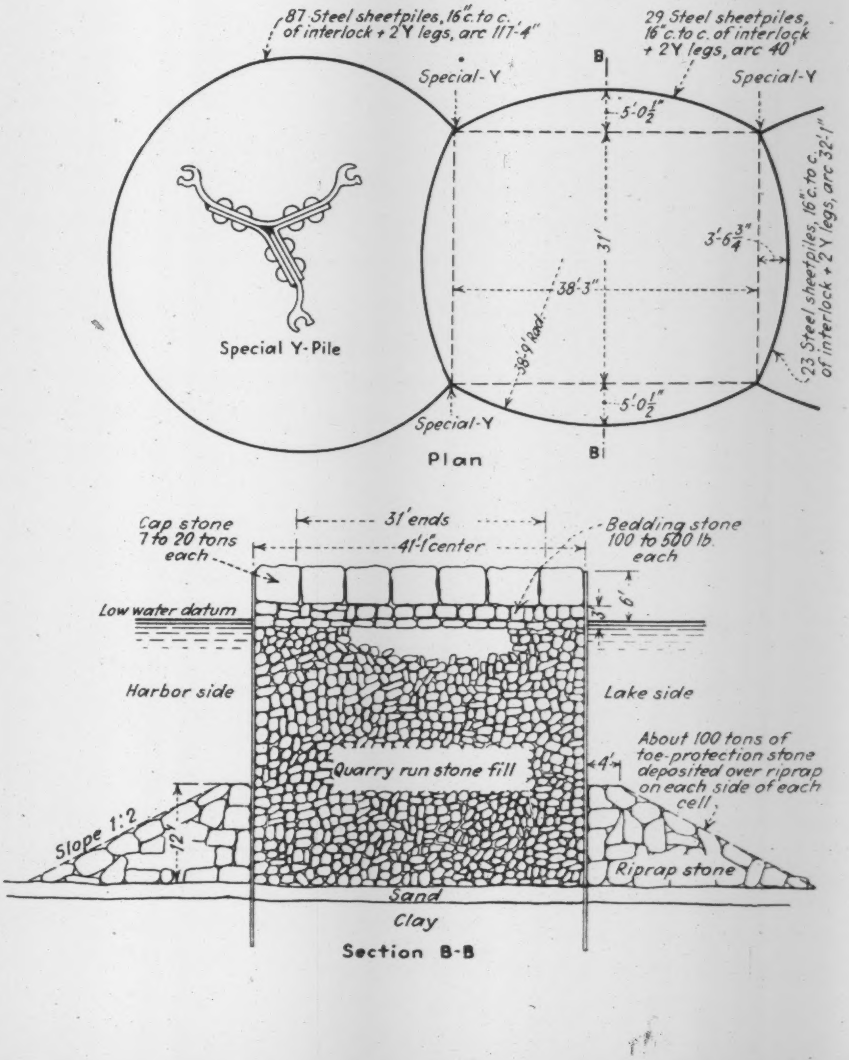
## Design reviewed

During the shutdown of construction due to the tugmen's strike, which continued from June 2 to Aug. 1, 1934, the question arose in the mind of the contractor of the stability of the structure as designed, and therefore he called the attention of the contracting officer to what he deemed "sub-surface or latent conditions at the site materially differing from those shown on the drawing or indicated in the specification." On Sept. 5, 1934, the contractor specifically advised the U. S. Engineer Office, "We have a firm conviction that this breakwater is weak in design, both during construction and after completion."

In view of this decided opinion of the contractor, the U. S. Engineer Officer retained the writer's firm as engineering consultants to make an investigation and report to him:

1. Are there any defects in the de-

**FIG. 4—CELLULAR STEEL BREAKWATER STRUCTURE for Calumet Harbor on Lake Michigan.**


87 Steel sheetpiles, 16" c. to c. of interlock + 2 Y legs, arc 117'-4"
29 Steel sheetpiles, 16" c. to c. of interlock + 2 Y legs, arc 40'
Special-Y
B
Special-Y
5'-0½"
3'-6¾"
31'
38'-3"
38'-9" Rad.
23 Steel sheetpiles, 16" c. to c. of interlock + 2 Y legs, arc 32'-1"
Special Y-Pile
Special-Y
5'-0½"
B
Plan
Cap stone 7 to 20 tons each
31' ends
41'-1" center
Bedding stone 100 to 500 lb. each
Low water datum
6'
3'
Harbor side
Lake side
Quarry run stone fill
4'
About 100 tons of toe-protection stone deposited over riprap on each side of each cell
Slope 1:2
12'
Riprap stone
Sand
Clay
Section B-B

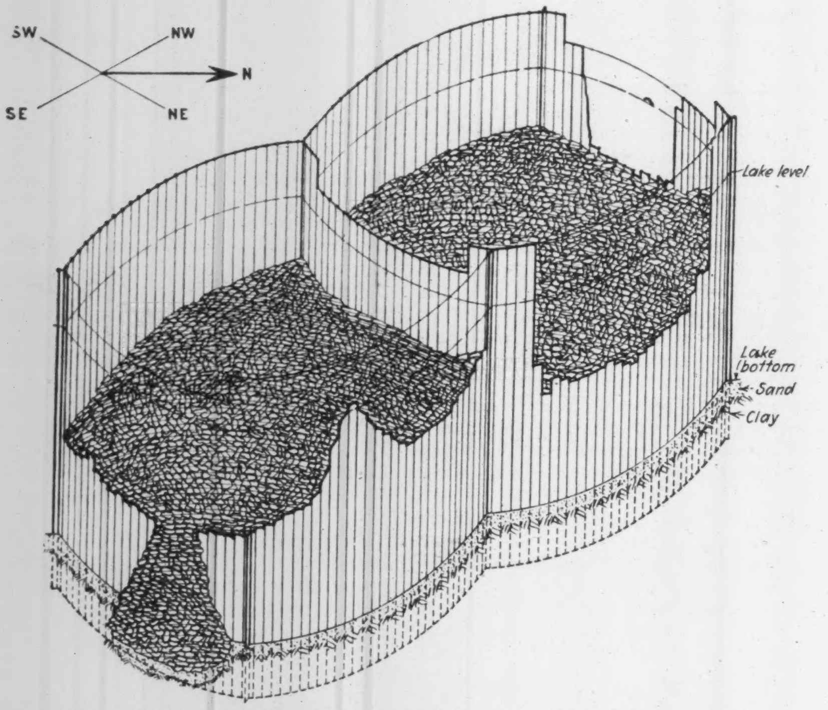

SW
NW
SE
NE
N
Lake level
Lake bottom
Sand
Clay

FIG. 5—WRECKED CELLS of Calumet breakwater, showing piles broken off by weaving under storm-wave action.

sign of the Calumet breakwater, as shown on the plans or described in the specifications, so serious that the United States would not be warranted to proceed with the construction under the present plans? This question to be considered from two angles: (*a*) the completed structure; (*b*) hazards during construction.

*2.* If the answer to the first question is yes, what are the minimum changes which you recommend to be made in order that the United States may proceed with minimum delay and expense and a reasonable assurance of obtaining a stable structure and with reasonable assurance that the contractor can successfully meet the hazards during the construction period?

The results of the investigation warranted the opinion that, "The design of the completed structure has no defects that would make it necessary or desirable to change the design, and the hazards during construction are not greater than those inherent in all such breakwater construction in similar locations subject to wave action." The investigation leading to the opinion included the following considerations:

*Wave Pressure*—The theory of wave pressures is one that has a long history. Rankine in 1858 developed an elaborate theory with corresponding formulas for determining the velocities and horizontal forces of waves. A paper by David A. Moliter, on the "Wave Pressures on Sea Walls and Breakwaters" was published in the May, 1934, *Proceedings* of the American Society of Civil Engineers. This paper brings together much of the best and most widely accepted theory at this time of wave action and includes the observations and dynamometer records of wave pressures made by Capt. D. D. Gaillard, Corps of Engineers, U. S. A., published in *Professional Papers No. 31*, U. S. Corps of Engineers, 1904.

Following the theoretical method formulated by Mr. Molitor, a wave-pressure curve was determined for the Calumet harbor structures, based on the maximum fetch of 300 miles and the maximum recorded wind velocity of 55 miles per hour, maintained for a period of two hours, for the location of Chicago. The effect of such a theoretical wave pressure on both the timber crib (Fig. 3) and the steel sheetpile (Fig. 4) structures at Calumet harbor is shown by Figs. 6 and 7.

As a check on this purely theoretical analysis, a similar wave-pressure curve was determined, based solely on the recorded observations of Captain Gaillard at Duluth harbor, also shown

FIG. 6—WAVE-PRESSURE CURVES and resultants as computed for timber-crib breakwater shown by Fig. 3.

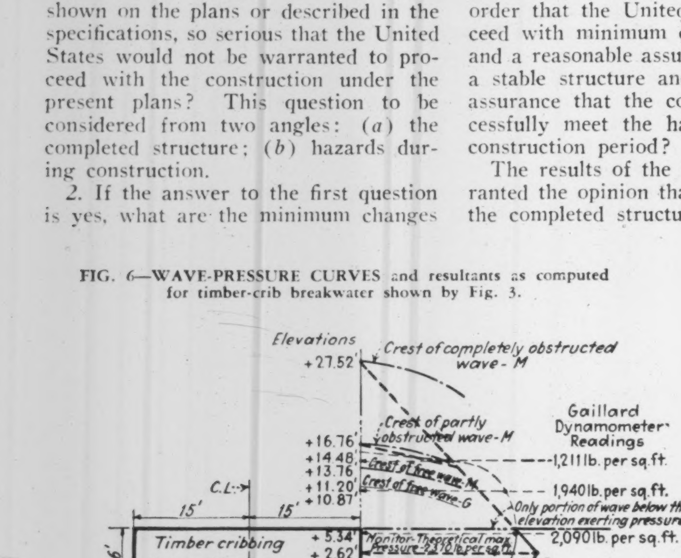
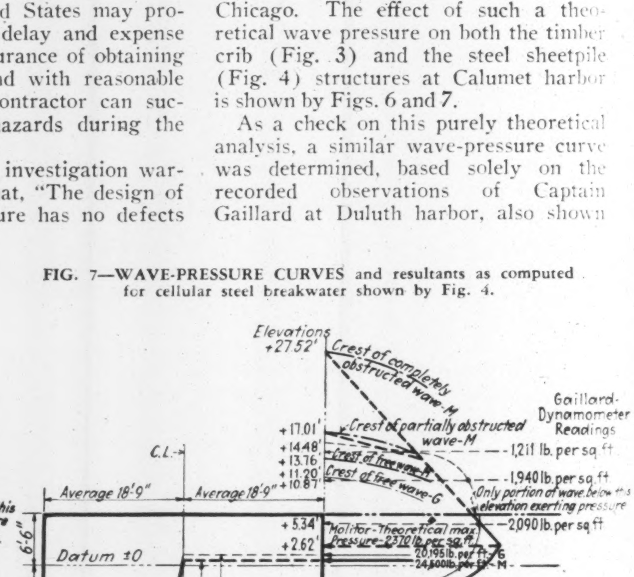

Elevations
+27.52'
Crest of completely obstructed wave - M
Gaillard Dynamometer Readings
Crest of partly obstructed wave-M
+16.76'
+14.48'
+13.76'
+11.20'
+10.87'
Crest of free wave-M
Crest of free wave-G
1,211 lb. per sq. ft.
1,940 lb. per sq. ft.
Only portion of wave below this elevation exerting pressure
2,090 lb. per sq. ft.
C.L.
15'
15'
6'
24'
Timber cribbing
Datum ±0
+5.34'
+2.62'
Monitor-Theoretical max. Pressure-2370 lb. per sq. ft.
19,150 lb. per ft.-G
23,480 lb. per ft.-M
Rock filled
-6.8'
-8.07'
Wave trough-G
Wave trough-M
6,380 lb. per sq. ft.-G
Resultant-M
Resultant-G
24.49-M
25.09-G
5.0'-G
3.1'
9.3'-G
11.9'-M
Middle third
Rock foundation
Lake bottom
10,400 lb. per sq. ft.-M
G
M
Wave Pressure Curves and Resultants
M- Molitor
G- Gaillard

FIG. 7—WAVE-PRESSURE CURVES and resultants as computed for cellular steel breakwater shown by Fig. 4.


Elevations
+27.52'
Crest of completely obstructed wave-M
Gaillard Dynamometer Readings
+17.01'
Crest of partially obstructed wave-M
+14.48'
+13.76'
+11.20'
+10.87'
Crest of free wave-M
Crest of free wave-G
1,211 lb. per sq. ft
1,940 lb. per sq. ft
Only portion of wave below this elevation exerting pressure
2,090 lb. per sq. ft
C.L.
Average 18'-9"
Average 18'-9"
6'-6"
Datum ±0
+5.34'
+2.62'
Molitor-Theoretical max. Pressure-2370 lb. per sq. ft.
20,195 lb. per ft.-G
24,500 lb. per ft.-M
Rock filled
-6.8'
-8.07'
Wave trough-G
Wave trough-M
32'
Steel piles
Steel piles
Resultant-M
Resultant-G
104,000 lb. per ft.
32.73'-M
33.36'-G
6,280 lb. per sq. ft.-M
Lake bottom
Middle third
12.23'-G
6.52'-G
11.04'-M
7.71'-M
8'
G
M
5,670 lb. per sq. ft.-G
Wave Pressure Curves and Resultants
M- Molitor
G- Gaillard

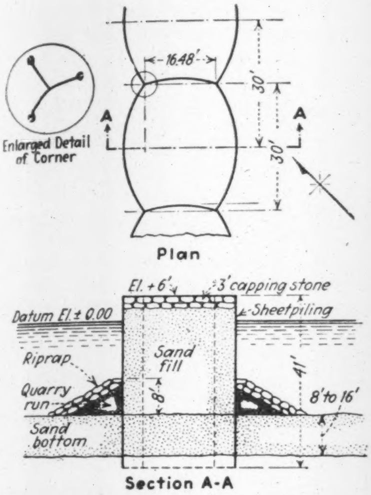


FIG. 8—CELLULAR STEEL breakwater at Indiana Harbor on Lake Michigan.

on Figs. 6 and 7. From these studies it was found that the theoretical wave force was considerably greater than the force of the maximum (18-ft.) wave measured by Captain Gaillard.

The above forces when applied to the old timber-crib structure (Fig. 3), which has successfully withstood the storms of 30 years, indicate that the structure would probably be wrecked by such forces except for the riprap protection. It not only has stood, but another breakwater of the same design, 5,400 ft. long, was built 50 years ago for the protection of Chicago harbor, and that too has stood all these years.

The steel cellular structure, owing to its greater width and weight and notwithstanding its greater height above base, has at least twice the stability of the timber-crib structure. Consequently the steel structure, disregarding any riprap protection and the 6-ft. penetration of the sheetpiling into the clay, is found to be stable when subjected to the same forces that were applied to the timber structure.

*Bursting Pressure* — The bursting pressure has to be considered in designing a cellular type of breakwater and adequate strength provided by the interlocks of the sheetpiling to resist this pressure. In computing the bursting pressure, the weight of the stone filling was taken as 100 lb. per cu.ft. The pressures were computed for a stone cap 4 ft. thick at 125 lb. per cu.ft., bedding stone 3 ft. thick at 100 lb. per cu.ft. dry, and 33.5 ft. of quarry-run stone at 100 lb. per cu.ft. dry. All the submerged stone was calculated at 63 lb. per cu.ft. The weight of the clay foundation submerged was taken to be 80 lb. per cu.ft., and the angle of repose 1 to 2. The water surface inside the breakwater was assumed at 4 ft. above datum, and outside, 8 ft. below datum.

The results of the calculations, disregarding the passive resistance of the clay outside the sheeting, showed a maximum tension on the sheeting interlocks of 5,358 lb. per lin.-in. for storm conditions, after riprap is in place, or 6,200 lb. per lin.-in. without riprap. The specifications required that the interlocks develop a minimum strength of 12,000 lb. per lin.-in., and the actual tests showed a minimum strength of over 16,000 lb. and an average strength of 21,900 lb. Consequently, the design was satisfactory as to the strength of the piling, as well as its stability against overturning.

## Other cellular breakwaters

In making the investigation of the stability of the steel cellular structure, two other steel cellular structures were considered. One was the breakwater (Fig. 8) built at Indiana harbor, Ind., on Lake Michigan, by the Indiana Steel Co. This structure is 480 ft. long and extends in a northwest direction. It consists of sixteen cells each 30 ft. long by a maximum width of 26 ft. The radii of the side walls are 26 ft.; the cross-walls have radii of 26 ft. and a chord of 16½ ft. Work on this structure was begun Sept. 29, 1933, and completed in November of that year.

After the first three cells had been completed and the fourth cell was started, a strong wind and rough lake caused the fourth cell to be abandoned, and it was lost. On Oct. 14, this cell was redriven, and by Nov. 1, fourteen cells had been completed. On Nov. 2 the wind shifted to the northwest before the next two cells were compltely filled. Cell No. 16, which was only partly filled, collapsed. Later this cell was redriven and filled. These cells are filled with sand instead of rock, but the upper 3 ft. is capping stone with the top at El.+6 ft. The lake bottom is sand at El. —19 ft. The clay is from 8 to 16 ft. below El. —19 ft. The steel piling penetrated 3 ft. into the clay. This structure had successfully stood for one year when the investigation was made and is now experiencing its second winter.

The other structure considered was the cellular-type steel sheetpile shore connection built at Port Washington Harbor, Wis., on Lake Michigan about 30 miles north of Milwaukee, for the U. S. government, on the design and under the direction of the U. S. Engineer Office at Milwaukee. This structure was then under construction, having been started in the spring of 1934. Its total designed length was 985 ft., made up of 22 cells each 44.79 ft. long. The lengths of the nearly straight cross-walls varied from a minimum of 8 ft. 3½ in. to a maximum of 16 ft. 2⅞ in. The top of the structure was to be at elevation 7½ ft. above datum. The lake bottom (sand) sloped from—7 to—15 ft. and clay from—10 to—21 ft. The piles were planned to be driven down to depths varying from —19 ft. to—27 ft. A typical plan and cross-section are shown in Fig. 9.

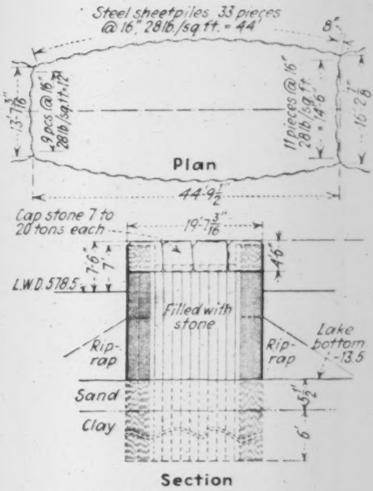

Steel sheetpiles 33 pieces @ 16", 28 lb./sq. ft. = 44'
8"
13'-7 3/16"
9 pcs. @ 16", 28 lb./sq. ft. = 12'
11 pieces @ 16" 28 lb./sq. ft. = 14'-6"
16'-2 7/8"
Plan
44'-9½"
Cap stone 7 to 20 tons each
19'-7 3/16"
7'-6"
7'
4'-6"
L.W.D. 578.5
Filled with stone
Rip-rap
Rip-rap
Lake bottom -13.5
Sand
Clay
5½'
6'
Section

FIG. 9—CELLULAR STEEL breakwater at Port Washington Harbor on Lake Michigan.

The Port Washington structure differs materially from the Calumet structure in that its side walls have very flat arcs, the radius being 85.07 ft. and the rise only 3 ft. This construction was also interrupted by the tugmen's strike between June 2 and Aug. 1, 1934, but before the work was shut down the completed cells had been filled with stone within 5 ft. of the tops of the sheetpiling. The structure stood for the two months with no damage other than some bulging of the side walls.

## Construction methods at Calumet

Construction was resumed at Calumet Harbor early in September, 1934, on the original plans and specifications. At first nothing was done about removing the first two partly filled cells, wrecked by storms after the shutdown of work in June 1934. These cells, Nos. 66 and 67, were at the center of the proposed line of 131 cells. Work was restarted on cell No. 64 north, and cell No. 69 south of the wrecked cells, leaving space for four cells between the two starting points and working both north and south. Between Sept. 10 and Nov. 15, 34 cells were driven and filled with stone in 58 days (excluding Sundays). After the two wrecked cells and stone filling had been removed, the four cells required to fill the gap were driven and filled, thus completing 38 of the required 131 cells, or approximately 30 per cent of the entire length of the proposed breakwater.

As required by the contract and specifications, the temporary end cells, Nos. 49 and 86, were made circular cells, having radii of 24 ft. 0½ in., and connecting with the adjacent typical four-sided cells.

After the work was resumed, the procedure of construction was to fill

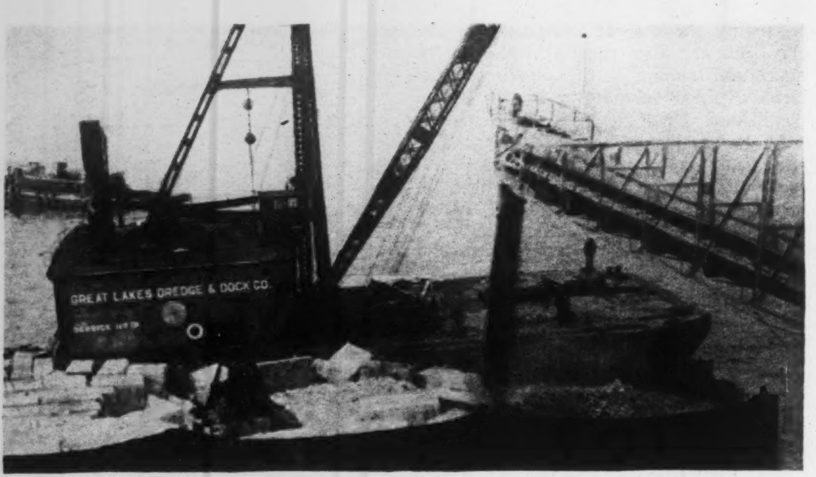


FIG. 10—STONE FILLING and capping of cells in Calumet Harbor breakwater.

each cell immediately after it was driven, with quarry-run stone to within about 7 ft. of the top, delivered by a belt conveyor from a stone boat as shown by the illustration, Fig. 10. By this method the entire quantity of filling stone for one cell, amounting to an average of 2,260 tons, could be placed in less than one hour. This would be followed by placing approximately 230 tons of bedding stone with as little delay as possible, and that in turn by 380 tons of capstone. Fig. 10 shows one cell being filled by stone from a conveyor through the openings in the templet, the templet being left in place to brace the piling until most of the filling stone was deposited. These templets were used repeatedly for successive cells. It also shows one cell in which the bedding stone has been deposited on top of the filling stone, and a few pieces of capstone placed. The third cell has been filled and nearly covered with capstone.

Fig. 1 shows a group of cells during construction and part of the contractor's equipment. The nearest cell has its templet still in place, and most of the sheetpiling has been driven to final position. One piledriver is driving piling in the cross-wall of this cell, and a second piledriver is driving piling in the side wall of the next cell. The cells beyond have all been filled with filling stone and some with bedding and capstone. Just beyond, or south of this group of cells, the piling of the west side walls and the intermediate cross-wall of the wrecked cells, Nos. 66 and 67, can be seen, and still farther south work is proceeding on the cells beyond No. 67, with two other equipment outfits. This photograph was taken on Oct. 2, 1934, at which date the progress report shows that nine cells had been driven and filled north of the gap and four cells south of the gap. On the scows moored alongside the derrick scows are frames of sheetpiling assembled in curved panels ready to be raised and placed for driving in additional cells. After these assembled panels are lowered in place, in their vertical positions around the templets, with their lower ends partly embedded in the sand or clay, and the separate Y's and closing piles are put in place to complete the cell walls, the temporary curved steel angles near the lower ends and the temporary curved channels near the upper ends of the panels are then removed. The curved angles are temporarily held in place by bolts in the sheetpiles, which engage the angles through slots in their vertical legs, so that the angles may be raised and freed from the bolts. The bolts holding the temporary curved channels near the upper ends of the panels are removed before the channels are taken off.

The riprap and toe-protection stone was deposited as soon as practicable after the driving and filling of each cell.

The filling stone was first handled from scows, by derricks, and required 18 to 24 hours to handle the filling stone for one cell in its clamshell bucket. This method was so slow that damage might (and did) result from waves, before the steel piling could be braced by the stone filling. This was one reason for changing to conveyor stone for filling the cells. As provided for in the specifications, such stone was to be so regarded as to produce the average specified weight of 90 to 100 lb. per cu.ft. and to range in size from ¼ in. to 14 in.

Conveyor stone was delivered by vessels designed for this purpose with cargo capacities of 12,000 to 13,000 tons. These ships have lengths up to 600 ft. over-all and beams up to 65 ft., and are completely equipped with turbo-electric motors and machinery for driving the conveyors. The conveyor stone was quarried at Roger City, Mich., on Lake Huron, and it was loaded by belt conveyors into continuous bins between decks on each side of the hull, extending the length of the vessel. After the vessel reached its destination, the stone was unloaded from the bins by belt conveyors, which delivered it to a vertical-lift bucket conveyor and thence to a horizontal belt conveyor carried by a revolving boom 200 ft. long and capable of delivering the stone on either side of the vessel as required. The maximum rate of delivery of the stone was over 2,500 tons per hour.

Due to the method of placing the cells in panels instead of singly and also to the requirement that the interlocking joints be welded together in the upper 2 ft., the distortion during filling was not as great as might be expected. The anticipated amount of distortion was difficult to predict, but the actual distortion was not such as to cause any concern. This distortion varies to some extent with the different makes of piles. The different steel companies furnished piling of almost identical cross-sectoin, but there are very slight differences in the interlocks.

After the bedding stone and capstone had been placed, it was found that an appreciable settlement of the stone filling had taken place. This settlement was especially noticeable after even a light storm, indicating that there is some movement of the cells themselves as well as the water in the cells, due to wave action, which causes a readjustment of the stone into a denser mass with a corresponding reduction of voids. The weight of a 38-ft. depth of stone gives an additional load of about 2,700 lb. per sq.ft. on the sand and clay of the lake bottom, over the normal load of about 2,000 lb. per sq.ft., due to the weight of the 32-ft. depth of water, or a total pressure of about 4,700 lb. per sq.ft. It is reasonable to expect the stone and sand to be forced down into the clay, or the clay and sand to find its way up into the voids between the stones, especially as the entire mass is under water. In some cases the settlement had been as much as 3 ft. up to the close of the work (Dec. 15, 1934). An inspection on Feb. 4, 1935, disclosed no notable change. It is probable that within a year or two it will be advisable to move the capstone and add bedding stone to bring the top of the capstone up to the desired elevation. The capstone was placed about 2 ft. above the desired elevation to allow for settlement.

## Bids and prices

Bids were received by the Engineer Officer on Sept. 25, 1933, for the construction of the breakwater for Calumet Harbor and were based on three alternate types of structures. The lowest bids were as follows:

| | Materials and Constr. at Site | Materials to Be Furnished by Gov't.* | Totals |
|---|---|---|---|
| A, steel cells, design A | $1,443,806 | $360,000 | $1,803,806 |
| B, steel cells, design B | 1,221,664 | 360,000 | 1,581,664 |
| B & C, ½ steel cells and ½ con caissons | 1,319,791 | 377,400 | 1,697,191 |
| C & R, ½ conc. caissons and ½ rubble mound | 1,465,442 | 197,400 | 1,662,842 |
| Rubble mound | 1,648,100 | ...... | 1,648,100 |

*Approximate figures for steel sheetpiling and concrete caissons.

The contract was let on the lowest bid, which was based on the use of design B for the entire structure. The estimated quantities required for the completed structure on design B of 131 cells, with an approximate total length of 5,025 ft., and the unit prices on which the contracts were let were:

| | | | |
|---|---|---|---|
| Steel sheetpiling, lb. | 14,683,000 | | |
| Special fabricated steel Y's., lb. | 936,000 | | |
| Total weight, lb. | 15,619,000 | | |
| Total Cost | | | $358,802 |
| Filling stone (at 95 lb. per cu. ft.) | 296,300 tons | @ $1.54 | $456,302 |
| Bedding stone | 30,000 " | @ 2.50 | 75,000 |
| Capstone | 49,900 " | @ 4.54 | 226,546 |
| Riprap stone | 113,000 " | @ 2.40 | 271,200 |
| Toe-protection stone | 25,100 " | @ 2.50 | 62,750 |
| Driving sheetpiling (per cell) | 131 cells | @ $991.35 | 129,866 |
| Total, exclusive of steel | | | $1,221,664 |
| Total, including steel | | | 1,580,467 |

Therefore, the cost per linear-foot for 5,025 ft. is $385, or $12,800 per cell, to which should be added the cost of engineering supervision, inspection and accounting by government officers and assistants.

The Calumet Harbor breakwater was designed by and was constructed under the supervision of the Corps of Engineers, U. S. A. The design was prepared under the direction of Col. W. G. Caples, then district engineer, Second Chicago District. The construction of the breakwater was supervised by the U. S. Engineer Office, Chicago, with Lieut-Col. Donald H. Connolly as district engineer.

Clarence R. Andrews, principal civil engineer, and H. A. Hoeing, senior engineer, were in charge of all design and construction in the lakes portion of the Chicago district, and G. T. Stewart, assistant engineer, was in immediate charge of the staff of engineers and inspectors at the site of the work.

The general contract for the breakwater construction was let to the Great Lakes Dredge & Dock Co., of Chicago, of which John F. Cushing is president and Wm. B. Murray superintendent. A. J. Meyer is manager, and John Hollowoed division engineer, South Chicago Division.

# Record Stream Discharges Feature New York Floods

## Cloudbursts in localized areas precipitated one-quarter of yearly rainfall in two days and large runoff swelled streams to record stages, causing estimated damage of $25,000,000

*Engineering News-Record Staff Report*

FLOODS of devastating proportions resulting from a series of scattered cloudburst storms swept across 7,000 square miles comprising ten counties in southern and central New York state on July 7 and 8, doing damage there and in northeastern Pennsylvania in excess of $25,000,000 and causing the loss of 55 lives.

Highways were washed out and railroad tracks and bridges destroyed at dozens of points. Towns throughout the inundated area were seriously endangered, as water-supply mains, gas and electric connections were swept away.

From general evidence and fragmentary reports it appears that the rainfall and high-river stages in most of the affected region were without precedent. Rainfall figures for many locations passed all records, but the variable character of the rain in localized areas makes maximum precipitation evaluation a difficult problem. There is a lack of adequate streamflow data at present. Runoff tabulation and flood-crest measurements will require careful study before comparisons can be made with records of former years.

### Soil-erosion severe

The greatest damage in the New York area occurred at Hornell, Ithaca, Binghamton and Watkins Glen (*ENR* July 11, 1935, p. 65, for map), where small streams became raging rivers within the course of a few hours. Ithaca and Binghamton were only partly flooded, the major portion of each city remaining unharmed. Hornell and Bath were almost completely under water. Trumansburg, north of Ithaca, and Hammondsport, west of Watkins Glen, were probably the worst hit in proportion to population.

The farm land of the region was severely damaged either by the loss of fertile soil or, in the case of bottom lands, by heavy deposits of silt swept down from adjacent hills. Preliminary estimates place the loss of top soil at 65,000,000 tons; sloping land in cultivation is believed to have been eroded to the extent of 65 tons of soil per acre.

### Rainfall reached 8 to 14 in.

T. E. Reed, meteorologist in charge of the U. S. Weather Bureau at Binghamton, reports that in Hector, 9 miles north of Watkins Glen, 14.23 in. of rain fell in 47 hours, and during this period there was one downpour of 10 in. in seven hours. At Cortland there was a fall of 8.16 in. in 24 hours, beginning Sunday night, July 7. At Ithaca the greatest rainfall in the history of the city occurred, with a fall of 8.12 in. in 36 hours, and 5.46 in. during the last twelve hours of this period. Throughout the area a precipitation of 1½ to 2 in. in 24 hours would represent heavy rainfall.

The distribution of the precipitation was no less abnormal than its amount. In some localities it reached cloudburst proportions, while near-by communities remained comparatively dry, as illustrated in the records for Norwich, where 8.17 in. fell in 38 hours, while Sherburne, only 10 miles farther north, reported but 1.74 in.

### Stream and lake stages

Intermittent rains had been experienced in the storm area for ten days previous to July 7, and the ground was fairly saturated. However, the intensity of storm rainfall was so great that the condition of the ground was a negligible factor in producing flood runoff. The streams normally carry a mere trickle of water in midsummer, but they quickly rose to abnormal heights. Preliminary estimates of maximum flows, made by the U. S. Geological survey, are shown in the accompanying table.

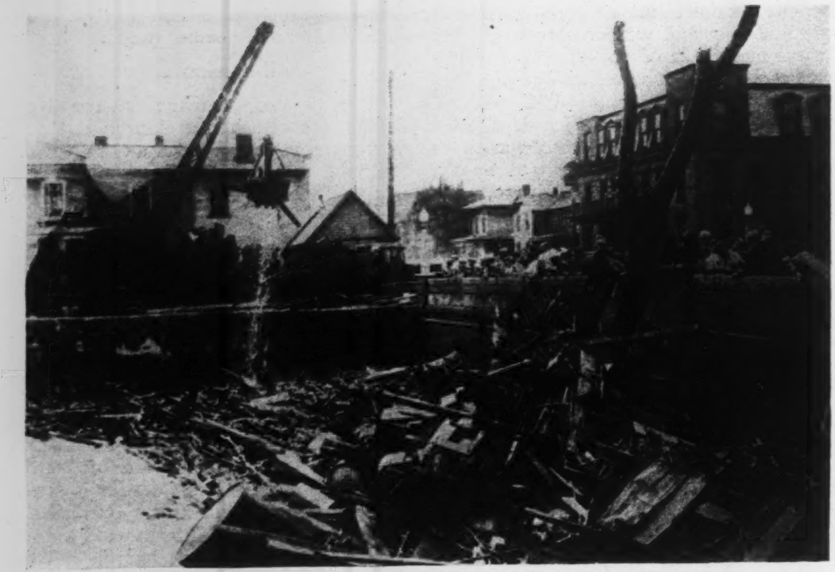
At Bainbridge, 35 miles northeast of Binghamton, the Susquehanna River rose in 24 hours from 0.2 to 14.2 ft. during a rainfall of 4.10 in., and an additional 3.5-ft. rise was noted with a

NEW YORK STREAM DISCHARGES COMPARED WITH PREVIOUS RECORDS

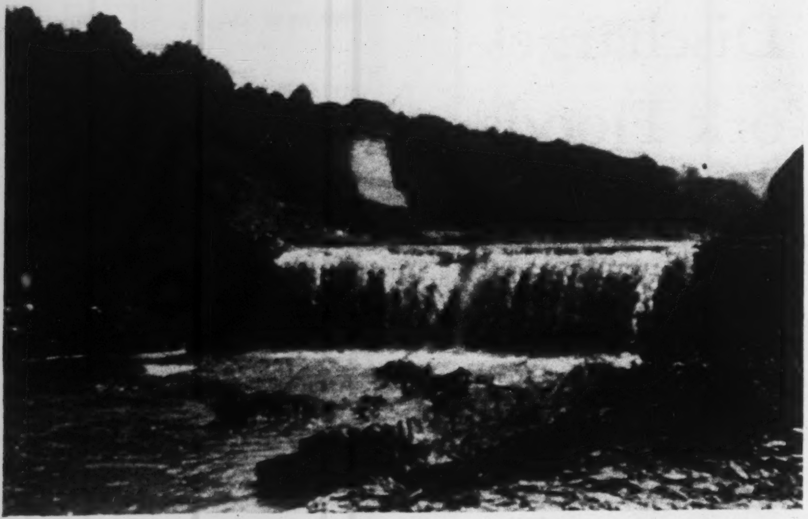
U. S. Geological Survey, Water Resources Branch, Albany, N. Y.

| Stream | Place | Discharge,* Sec.-Ft. | Drainage Area, Sq. Miles | Sec.-Ft. Per Sq. Mile | Previous High Discharge, Sec.-Ft. | Date | Length of Record |
|---|---|---|---|---|---|---|---|
| Canaseraga Creek | Dansville | 10,000 | 153 | 65 | 6,900 | 11/30/27 | 1910–12<br>1915–17<br>1919–34 |
| Susquehanna R. | Conklin | 41,600 | 2,240 | 19 | 52,000 | 3/28/13 | 1912–34 |
| Tioughnioga R. | Itaska | 50,000 | 735 | 68 | 12,500 | 4/ 1/34 | 1929–34 |
| Chenango R. | Chenango Forks | 80,000 | 1,490 | 54 | 35,500 | 3/27/13 | 1912–34 |
| Cohocton R. | Campbell | 20,000 | 472 | 42 | 12,900 | 12/ 1/27 | 1918–34 |
| Allegheny R. | Red House | 3,710 | 1,694 | 2 | 41,000 | 3/ 2/10 | 1903–34 |
| W. Br. Delaware R. | Hale Eddy | 17,200 | 593 | 29 | 26,500 | 9/30/24 | 1912–34 |
| E. Br. Delaware R. | Fishs Eddy | 3,180 | 783 | 4 | 53,300 | 8/24/33 | 1912–34 |
| Delaware R. | Port Jervis | 30,100 | 3,076 | 10 | 155,000 | 10/10/03 | 1904–34 |
| Chemung R. | Chemung | 80,000 | 2,530 | 20 | 67,000 | 3/15/18 | 1903–34 |

*Preliminary estimates subject to revision.



**UNDER THIS DEBRIS** at Hornell's River St. Bridge is the wrecked steel-truss span of the East Elm St. Bridge. The concrete channel walls that confine the Canisteo River through the city protected adjacent property from complete destruction.



**BREACH IN EARTH DAM** that formed a water-supply reservoir for the city of Hornell. The pipe lines whose breaking deprived the city of water are in the shadow near the bottom of the left abutment. Upstream is an old intake tower.

further precipitation of 0.9 in. The records of the Weather Bureau showed these changes in the Susquehanna and Chenango Rivers, which meet at Binghamton: on the morning of July 7 the Susquehanna had a gage height of 2.08 ft. and the Chenango 1.39 ft.; at 5 a.m. on July 8 no rise was evident, but three hours later the Susquehanna gage was up to 9.60 ft. and the Chenango reading stood at 13.50. This latter stream reached a crest of 20.10 ft. at 3 p.m. of that day, and from then on it receded. The Susquehanna advanced to 17.80 ft. at 2 a.m. on July 9, but as the Chenango had already dropped below this level the unique phenomenon of a river running uphill occurred as the Susquehanna waters reversed the flow in the Chenango River. The illusion was strengthened when, at one point, the waters were forced upstream over a low dam.

Lakes Seneca and Cayuga, the two largest bodies of water of the Finger Lakes group, experienced considerable rises. The former, draining an area of 700 square miles, rose some 3 ft. in less than 24 hours. A new official lake-level record was set on Cayuga with its rise of 2.68 ft. since July 1, making it now 4½ ft. above normal level. At this elevation it was reported to be passing over the New York Barge Canal locks at the northern end. Floodgates on the Baldwinsville Dam at this end of the lake were lifted to their maximum opening of 5 ft., and state engineers directed that they be cut by acetylene torches to give 2-ft. additional opening. This added head was sufficient to raise the level of the Seneca River, outlet of Lake Onondaga, so that here, too, water ran upstream.

## Hornell without water supply

Hornell, farthest west of the cities in the southern tier counties to be ravaged with floods, suffered a major disaster when almost the entire 1,500 acres of its area was inundated. The construction with state funds of control dikes along the Canisteo River several years ago, at a cost of $350,000, served to retain a great part of the rocks, trees and other debris in the regular stream channel, thus preventing far greater destruction than was experienced. These dikes were built considerably higher than the normal spring floods, but on this occasion they were topped by 4 feet. Two flood crests swept through the city, the first occurring early on the morning of July 8, when Dam No. 1 of the city water supply gave way, sending more than 50,000,000 gal. into the already swollen streams. The second flood occurred at mid-day from heavy runoff in the hills surrounding the city, bringing the Canisteo River and Canacadeo Creek, which run through the town, more than 14 ft. above their normal levels. Considerable alarm was felt on the following day, when two cloudbursts above Hornell brought the rivers to their former high levels.

Although no runoff determinations were made, an idea of the volume of water that was flowing in this area is obtained from the measurement of a 4.25-ft. head over the 135-ft. circular spillway of No. 3 Dam, which impounds 150,000,000 gal. This dam, built in 1931, was so designed that a 5-ft. head on the spillway, estimated to be a 500-year possibility, would wash out an embankment section on the shore edge of the spillway. This safety device was within 0.75 ft. of being called into action.

The city's water supply reaches the filter plant from four reservoirs by means of 16-in. and 24-in. cast-iron pipes carried along the streambed for some distance. The flood from the dam failure broke both of these lines in several places. Thus, although the distribution system within the city remained operative, the treatment plant could receive no supply. To meet the emergency, the city forces set about building a temporary line from a hillside spring pond to the filter house. This pond, a 10-ft. hole in high ground from which springs are bubbling, dug last year to forestall a drought shortage, can deliver more than 1,000,000 gal. per day. Pending completion of the temporary line, the 16,000 residents of Hornell were put on a rationed water supply delivered from milk cans four times daily by department of public works trucks. This water is obtained from a deep well used by a local brewery.

Residential and business damage at Hornell was estimated at more than $1,000,000, but only two lives were lost. The Erie Railroad's large shop and yard facilities at Hornell were disabled for several days, the shops being filled with several feet of silt and the tracks in some places scoured out to a depth of more than 10 ft. Traffic between New York and Chicago on this line was interrupted for two days. Gravel outwash from the hills buried the tracks in spots to a depth of more than 8 ft.

Of the twelve bridges crossing streams in Hornell, a 100-ft. truss span at Genesee St. was washed out, and a similar structure at Church St. was badly damaged. On the Canisteo River the East Elm St. bridge was floated from its piers and carried downstream to jam against the River St. twin-arch concrete span. Although a thorough survey has not been made, city officials believe that this blockade with its resulting pressure of water may have moved and weakened the structure. Two 60-ft. trusses over Crosby Creek were badly twisted and damaged, and a third was carried out completely. The waterways between the piers of the Erie Railroad bridge were so filled with logs, house wreckage, and other debris that two days of work with dynamite and cranes were required to clean out the jam.

The estimated cost of bridge replacement throughout the city is put at $180,000. Damage to the water system is estimated at $80,000, but it is yet to be determined whether or not there are any major breaks within the city limits. Sewer repairs will come to $15,000. Although the Imhoff tanks in the sewage-disposal plant were flooded, no water entered or damaged the glass-covered sludge-drying beds. Street and pavement damage is estimated at $100,000.

### City engineer directing repairs

F. Hurd Robinson, city engineer for the past 22 years, is directing the herculean task of rehabilitating the city, and he believes that in 30 days all but the major repairs will be completed. Displaying a genius for coordination, he organized within two days a crew of 500 men, 100 trucks and four cranes into units that are cleaning hundreds of tons of dirt from the streets, pumping cellars, removing wreckage, supplying water and filling in washouts. This clean-up program will cost more than $200,000.

The failure of the water supply is considered to be such a menace to health that the city officials are providing free inoculation against typhoid and are urging all the residents to take advantage of this protection. To combat the huge clouds of dust that are now rising from the sun-baked silt, the streets are being sprinkled several times a day by converted asphalt-distributor trucks; calcium chloride is also applied along the curbs.

### Ithaca dams in no danger

Ravaged by a severe flood in 1906, Ithaca made channel improvements in its streams which for 29 years served to keep these streams within their banks during spring periods of high water. The improvements consisted of deepening and widening Cayuga Inlet and building levees along the three streams that traverse the city. The record rainfall of 8.12 in. in 36 hours, beginning in the early morning hours of July 7, ran off in sheets from the steeply sloped hills around the city, and quickly filled the watercourses to overflowing, as they poured into Lake Cayuga. The southwest section of the city, bordering Cayuga Inlet, was completely flooded and filled with debris; 200 families were driven from their homes, and a part of the business section was damaged through inundation. Residential property damage, totaling $150,000, was due largely to flood waves in the small watercourses, which overtopped their banks to form new channels.

Of the 34 street bridges in Ithaca, most of which were under water, only one small foot bridge was lost. However, the 135-ft. Lehigh Valley Railroad bridge, constructed in 1903 and for many years the longest plate-girder structure in the East, was washed out completely.

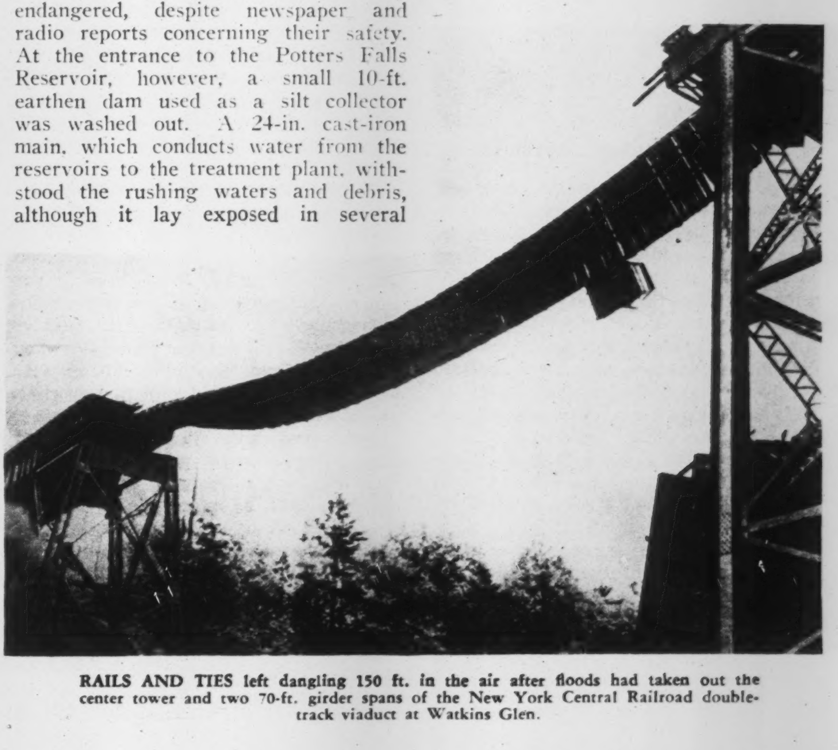
The three dams which serve to store the city water supply were at no time endangered, despite newspaper and radio reports concerning their safety. At the entrance to the Potters Falls Reservoir, however, a small 10-ft. earthen dam used as a silt collector was washed out. A 24-in. cast-iron main, which conducts water from the reservoirs to the treatment plant, withstood the rushing waters and debris, although it lay exposed in several places. The filter system at the treatment works was protected by a series of culverts around the plant and, although turbidity increased by about 7,000 p.p.m., which was combated with extra heavy doses of alum, pure and clear water was available at all times.

The sewage-treatment plant, on the banks of Cayuga Inlet near the lake, was out of service for 24 hours but was not damaged. Complete inundation of the municipal airport (*ENR*, April 18, 1935) on the banks of Lake Cayuga caused no visible injuries to the paved runways or other appurtenances. The newly opened municipal golf links near the airport was badly eroded, with most of the seeding and fill washed out. The salt-stabilized road adjoining the airport, flooded to a depth of several feet, was free from ruts shortly after the waters receded, and presented a well-consolidated surface despite a heavy flow of traffic.

Harry W. Eustance, city engineer of Ithaca, is coping with the problem of rehabilitation, using an expanded engineering personnel to direct the activity of 200 men supplied by the TERA and 50 boys from a local CCC camp. His estimate of damage to public property totaled $143,000, including bridges, $5,000; dikes $53,000; streets, $32,000; water mains $10,000; sewers, $10,000 and parks, $33,000.

### Binghamton loses large bridge

Fifteen hundred people were made homeless in Binghamton when the Chenango River overtopped a concrete dike and spread through the northwestern part of this city. Built in



**RAILS AND TIES** left dangling 150 ft. in the air after floods had taken out the center tower and two 70-ft. girder spans of the New York Central Railroad double-track viaduct at Watkins Glen.

1910, the dike was designed to be 4 ft. higher than any previous flood level recorded at that time.

Although both the Chenango and the Susquehanna Rivers flow through Binghamton, joining near the center of town, it was the first and smaller stream which caused the greatest destruction. Sweeping along at an estimated velocity of 15 m.p.h., it carried away the plate-girder bridge which spans the Chenango at Ferry St., one of the city's principal thoroughfares. This four-span structure, 354 ft. long, was built in 1889 and had recently been renovated and strengthened. Seven other bridges in the city were undisturbed.

Adding to the general havoc was the loss of three of the six water mains which crossed the rivers. One of these, a 20-in. pipe attached to the Ferry St. bridge, went out with the structure; the other two, laid in the river bed, were scoured out. Certain sections of the city were without water for two days until auxiliary connections were made. Before this was accomplished, a fire broke out in the flooded district, which threatened the destruction of the entire first ward.

J. A. Giles, city engineer, estimated the damages in the city to exceed three-quarters of a million. A temporary bridge, to replace the Ferry St. crossing, will cost $50,000.

Hammondsport, center of the Keuka Lake grape belt, with a population of 1300, was ravaged by two floods on successive days. The high waters ripped Glen Creek from the man-made channel, built several years ago to divert it past the business district, and sent it back into the main street, its original channel. A winery storage warehouse was toppled over and 1,200 casks of brandy, each weighing 500 lb., became animated battering rams in the swirling waters.

### State park damage enormous

The Watkins Glen State Park was declared to be practically in ruins in a statement issued by Lithgow Osborne, conservation commissioner of New York. All of the elaborate improvements placed in this beauty spot during the last generation were either destroyed or damaged to the extent that complete renovation will be necessary. Two spans and the central tower of the New York Central Railroad viaduct which crosses Glen Creek in the park, were washed out along with many highway and foot bridges. The railroad bridge built in 1867 consisted of 30-ft. tower spans and 60 and 70-ft. plate girder spans. The center tower (150 ft. high) located in the center of the stream (for reasons unknown to the present generation of New York Central engineers) was taken out with the adjacent 70-ft spans, leaving one track dangling as shown in the accompanying view. Traffic on this branch line will be discontinued for a month while the washed out spans are being replaced with a single 110 ft. span resting on new 30 ft. towers erected as extensions of the present 30 ft. towers on the edge of the gorge. The new foundations are to be placed 30 ft. above the bottom of the gorge, slightly above last week's high water mark.

In other park areas of the Finger Lake district, notably Enfield Glen between Ithaca and Elmira, the work of years was obliterated in a few hours. This is particularly evident in the landscaping details, which have required a long period of time for their perfection.

At Bath all the rearing ponds in the state fish hatchery were washed out with the loss of several hundred tho sand brook and lake trout.

To take care of the 3,000 perso temporarily made homeless, and combat the danger of disease and fi the Red Cross joined forces with s New York state departments, the CC and a host of lesser agencies. Th state TERA appropriated $375,00 with which 25,000 men were i mediately put to work removing debr and clearing roads.

Reports have not yet been receive from all of the ten counties in th flood-stricken area. Some of those re porting and their estimated damage are Broome, $3,000,000; Cortland, $1,550,000; Delaware, $1,000,000; and Tioga, $1,000,000.

# Fire-Service Charges Analyzed at N.E.W.W.A. Meeting

MARKED DIFFERENCES between the charges of municipally owned and private waterworks systems in New England and the desirability of unifying such tariffs were featured in a paper by H. E. Halpin before the New England Water Works Association at its spring meeting. Results of an investigation of 177 systems were given. A total of 84 municipal waterworks receive no revenue for public fire-protection service, but practically all the public systems in Maine are exceptions. Many of the waterworks in Maine are water districts set up as corporations. Their finances are separated from municipal funds, and their rates are under the jurisdiction of the state public utilities commission.

The municipal waterworks that enjoy payment for public fire protection receive an annual per-capita revenue of $0.66 compared with $0.88 for privately owned works, with an average for all works of $0.76. The total cost of fire-protection service, including fixed and operating charges, is from 20 to 35 per cent of the gross annual revenue, comparing the larger cities and the smaller communities. The municipally owned systems receive for public fire service an average of 12 per cent of the gross annual revenue, private systems receive 15 per cent, and all average 13 per cent. The average percentage of no New England state is as great as that which should be received, in the author's opinion, based on the average cost of fire-protection service. No states are known to the author, except Wisconsin, that have established rates offering adequate compensation for the costs of public fire protection.

The average annual rate per hydrant received by municipally owned waterworks in New England is $28, and by private works $47. The average of all is $37. The range in hydrant rentals is from $5 to $133. Most of the rates have been arbitrarily taken to approximate "the average near-by rate," or to guarantee a revenue to meet certain fixed charges. Few have been established with due consideration to the cost of service.

Practically all the works receive revenue under a hydrant-rental basis. A lump sum for fire protection is received by a limited number, and a few, notably in Connecticut, use the inch-foot basis. No works use the mile-of-pipe basis. It was said that considerable progress could be made by replacing the hydrant method of charging by that based on the inch-foot units of pipe in the distribution systems with an annual allowance per hydrant for maintenance. The inch-foot method is considered the most equitable and is coming into greater use. The number of inch-foot units is obtained by multiplying the linear-feet of pipe of each size by the diameter in inches. The rate of those few works which reported on this basis ranged from $0.0044 per in.-ft. plus $10 per hydrant to $0.01 per in.-ft. plus $12 per hydrant.

General practice in New England is for the user to pay the entire cost of the connection from the main to the property line. Any investment by the water department in the connection from the main should, in the author's opinion, be reflected in a proper charge to cover interest, depreciation, and maintenance. Two-thirds of the municipal systems make no charge annually for connections serving automatic sprinklers, private fire hydrants and standpipes. Practically all the private waterworks make such charges.

Twenty-seven per cent of the systems

reporting use the flat charge; 16 per cent use the size-of-connection basis; and the rest base their charges on outlets, either on hydrants alone, sprinkler heads or combinations. The author maintained that present practices should be simplified by using fewer charges. Charges based on the size of the connection range from $27.50 to $474 per year for 6-in.-diameter pipe, and on sprinkler heads only from $0.02 to $0.18 each per year. Charges of private systems are substantially greater than municipal systems.

# *Letters to the Editor*

## Flow Over Rounded Crests

Sir—Professor J. J. Doland, writing in your issue of April 18, 1935, has asked for substantiation of the exponential formula that he presented. The writer had devised an almost identical formula for spillway crests the shape of which corresponds to the under side of the nappe from a sharp-crested weir. The formula takes the form:

$$Q = \frac{CLH^{3/2 + n}}{H_d^{\,n}}$$

$H$ is the head on the crest corrected for velocity of approach, $Q$ and $L$ have their conventional significance, $C$ and $n$ are experimental coefficient and exponent respectively, and $H_d$ is the "design head" or the head above the spillway crest for which the crest theoretically takes the shape of the under side of the nappe. This would ordinarily be the maximum head expected, but as Messrs. Rouse and Reid have pointed out (*Civil Engineering*, January, 1935) the design head can be exceeded to quite a degree without danger of the nappe springing clear if care is taken with the spillway shape in the vicinity of the crest.

The use of the design head rather than the radius of curvature of the crest has the practical advantage in preliminary studies of being known without actually laying out the structure. Furthermore, a hydraulically correct spillway design will not have a constant radius of curvature. In the vicinity of such a crest, $R$ is roughly 0.7 $H_d$. However, for circular crests Professor Doland's formula is of merit.

The coefficient $C$ appears to vary inversely with the ratio $H_d/D$, where $D$ is the approach channel depth at the design head and $H_d$ includes the head due to velocity of approach. However, there is insufficient evidence for such a conclusion, and it may well be that slight discrepancies in the shape of the dam near the crest are of greater significance.

The writer believes the formula

$$Q = \frac{3.97\ LG^{1.62}}{H_d^{\,0.12}}$$

substantially represents the flow over spillway crests that are of the above design.

No doubt there is much information available in support or denial of these conclusions, and it would be of value to have such made known. Especially desirable would be data regarding the variation, if any, of the coefficient $C$ with the ratio $H_d/D$.

ROSS N. BRUDENELL,
Tennessee Valley Authority.

Knoxville, Tenn.,
May 15, 1935.

## Sewer Lining Resists Gas Attack

Sir—The writer has noted with interest an article "Sewer Lining Tested for Corrosion Resistance" by H. G. Smith, of Los Angeles (*ENR*, March 28, 1935, p. 444).

The work that Los Angeles is doing in weeding out unsuitable and undesirable sewer-lining materials is of great importance to engineers. It is to be hoped that eventually an easily applied solution of some character will be found which will have all the resisting qualities and long life desired in such a material.

An article on "Sewer Lining Experiments for Los Angeles County," describing an experimental line built by the county sanitation districts at the city's Hyperion sewage-disposal plant appeared in *ENR*, Jan. 7, 1926, p. 18. In the experimental sewer used there different lining materials were used, which, from analysis, synthetic tests, etc., appeared suited to the districts' needs. The test under severe operating conditions was made to determine which of those selected would survive.

As a result, the sanitation districts lined the larger-diameter sewers with the materials, in the manner described in *ENR*, Sept. 1, 1927, p. 346, using vitrified-clay liner blocks and a sand-sulphur-silica mixture similar in character to that described by Mr. Smith.

On Nov. 17, 1933, the experimental sewer of Hyperion again was inspected after an operation period of about eight years. Following are the observations of the writer at that time: (See reference for description of sections.)

*Section 1* appeared to be in good condition as to the segmental blocks, but the jointing material above the flow line was very soft and largely displaced in the transverse joints; whether or not there has been any cement-mortar decomposition behind the inner plates was not determined.

*Section 2* was in good condition below the flow line, while at and above the flow line there was some slight evidence of a leaching out of the sand-sulphur-silica mixture used to seal the joints between pipe liner blocks.

*Section 3* was in good condition below the water line, but above the water line the material had leached out of the transverse joints, although the liner plates were still in place; whether or not concrete disintegration had taken place behind the blocks was not determined.

*Section 4* was of particular interest in that in this section were three 36-in. pipes that had been completely lined with a mixture of sand-sulphur-silica, 0.4, 0.6 and 1 in. thickness for pipes 3, 2 and 1, respectively. The important observation here was that at about the flow line of sewage in the pipe the sand-sulphur-silica had completely disappeared back to the concrete in pipes 3 and 2, and that wherever in the periphery of the pipe the lining material had not been thoroughly bonded to the concrete, the concrete back of the lining material was destroyed for a depth varying from $\frac{1}{2}$ to $\frac{3}{4}$ in.

The most logical explanation of why the sand-sulphur-silica lining at the flow line has disappeared seems to be that at that point there exists the most favorable opportunity and environment for the activity of sulphur oxidizing bacteria. The rate of removal in this instance was about 1/16 in. per year.

While the rate of deterioration for the sand-sulphur-silica mixture in Section 4 appeared to be quite rapid, that in Section 2, where the sulphur-silica mixture had been used to fill the joints between liner blocks, was slow, the indication being that in the closely confined crack between liner blocks there did not exist those conditions admitting of rapid oxidation of the sulphur.

*Section 5* was in poor condition, the thin washcoat of sand, sulphur and carbon having almost completely disappeared above the flow line, exposing the concrete pipe to rapid disintegration.

*Sections 6 and 7* were in good condition below the flow line, but above the flow line the tar and asphaltum base materials were largely gone, and the pipe here showed customary evidence of deterioration.

The sewers lined in the manner described in *ENR*, Sept. 1, 1927, have been in operation for seven years. Inspection of the lines discloses that where the concrete is protected in the manner described, with clay liner blocks and joints filled with sand-sulphur-silica mixture, it is in good condition. The sulphur-silica mixture between liner blocks shows some slight evidence of destruction on the exposed interior surface, and while no accurate measurements have been made to indicate the annual rate of oxidation of the sulphur, it might be stated as approximately 1/100 in. at locations observed.

A. M. RAWN,
Asst. Chief Engineer,
County Sanitation Districts
of Los Angeles County.

Los Angeles, Cal.,
April 9, 1935.

# ENGINEERING NEWS-RECORD

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EDITOR—F. E. Schmitt. EDITORIAL STAFF—V. T. Boughton, N. A. Bowers, W. G. Bowman, W. W. DeBerard, C. S. Hill, H. W. Richardson.

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July 18, 1935

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## *Opportunity to Learn*

RECENTLY several light earthquake shocks have been felt in central California. They come at a time when the two great bridges crossing the Bay and the Golden Gate are under construction, their towers completed to full height and cable work in progress. It is an unusually favorable time to observe the movements, if any, of these high structures under the impulse of slight earth disturbances, as an invaluable means for predicting their behavior under the more severe shocks that may be expected at some time in the future. It would be necessary for the purpose, of course, to set up seismographic observing instruments on the towers, and perhaps to design new forms of instrument for the quite unusual service involved in recording the oscillation of these high and slender structures, especially if the movements be of any considerable magnitude. But the results would furnish information now totally lacking and would provide a conclusive answer to many questions that persist in spite of the able and skillful analyses of the engineers. With such data at their command, the builders of the bridges could turn over the completed structures to public use, assured of their permanence.

## *Some Progress*

RELIABLE REPORTS state that few men have as yet returned to work under the terms of the four-billion-dollar program. It is now more than two weeks after the beginning of the fiscal year in which the effort of the campaign was to be concentrated, and the campaign ought to be in full swing; the actual fact is that it has not yet made a start. A glance at what has been done in making preparations, in formulating and announcing policies, shows that as a whole the enterprise is yet in its early stages, and that months more will elapse before there can be substantial results. In fact, official announcements promise nothing more favorable than that work will be well begun in two or three months. Fortunately, the news of the past week also brings a few favorable signs. The drive on PWA non-federal projects is getting in motion; restrictive regulations on cost of project, on selection of labor, on wage rates and on purchase methods, are being relaxed and made workable; grade-crossings and roads are about to get going with energy. The progress now in sight gives promise of busy activity by fall, and in the meantime the way is clearing rapidly for effective cooperation by local authorities. Finally, all developments are perforce turning the program away from its predominant welfare aspect and giving it an increasingly constructive character.

## *Geography or Habit?*

BACK IN 1927 a series of violent rainstorms over northern Vermont produced destructive floods that wrecked dams and roads and did ten to twenty millions of damage. The state being small and by no means rich, the President offered the Governor federal help in repairing the losses. The Governor promptly answered that Vermont could take care of its own. That was eight years ago. A similar flood visitation last week struck the large and wealthy state of New York, doing much damage in the aggregate but far less in proportion to the population and resources of the state. Governor Lehman has not waited for an offer from Washington but has appealed to the President, saying, "The need for federal assistance to stricken farmers is very great." His appeal carries not a hint that the Empire State is able to take care of its own. Why the difference from Vermont's attitude? Are self-reliance and self-help matters of geography or of character? Or have we learned too well the bad habit of leaning on others?

## *A State Obligation*

GOING FROM PSYCHOLOGY to physical fact, we find a record-breaking intensity of rainfall in the New York storm (or perhaps chain of storms). The data are yet incomplete, but such as are available indicate that all prior rainfall records were exceeded at many points and that stream discharges at a number of places went far beyond the highest known. Rainfall intensities of 9 to 14 in. in two days are exceedingly high for the weather province involved. On the other hand, they are not so far beyond reasonable possibility as to give the result the aspect of an "act of God." This being so it should be noted that flood risks have so far received little or no attention on the part of the state. Even studies of what flood intensities are likely to occur, and how often they may occur, are virtually absent. The old doctrine that these matters are the concern of local authorities has evidently held sway in New York, even though all modern experience teaches that they go beyond the power of local governments. Major streams exceed the scope of county control, and in the mountain areas even minor rivulets present problems beyond the technical and financial capacity of the local authorities. These conditions make flood problems inherently a state concern, but the obligation has been neglected almost everywhere in the past and still is neglected by most states. It is to be hoped that last week's floods will serve as a reminder to state officials and legislators everywhere of the responsibilities imposed upon them by physical conditions within their areas.

## *Protecting an Investment*

MODERN TRUNK HIGHWAYS represent a large investment of public funds from which the public does not always receive maximum return, and strangely enough in many cases this is so because the investment is not large enough. For example, every bridge narrower than the approach highways, every sharp curve, every traffic light cuts down the capacity of highways very greatly. Elimination of such obstructions presents an economic problem of major proportions, as witness the million-dollar bridges described in this issue which eliminate bottlenecks on the modern shore highways north and south of Boston. When the highways were built, the decision was made to defer replacing the old bridges because of the large additional cost. But as long as the narrow bridges were allowed to remain, the investment in modern approach roads could not be utilized fully because the traffic routes could not be used to anywhere near capacity. One of the most difficult questions that highway engineers have to answer concerns the justifiable investment on a given route. But it is paramount that whatever investment is made be considered in the light of the traffic capacity of the road as a whole. Elimination of obstructions to the free flow of traffic, as Massachusetts has accomplished with the aid of the federal government, is one of the best available ways to protect the necessarily large investment in modern roads.

## Growing Interest

IT IS GRATIFYING to note that there is a steady increase of interest, though as yet painfully slow, in the problems of shore protection. The meeting of the American Shore and Beach Preservation Association last week testified to progress toward more thorough study of the complex questions which the subject presents, but it also revealed that the roll of states actively concerned in safeguarding their shores against destruction is still limited to the four or five pioneers.

Gen. Edward M. Markham's picturesque complaint that for lack of customers the Beach Erosion Board is not doing much business aptly characterizes the situation. The federal government has set up an able and experienced body to serve as adviser to states who contribute to its studies, but that body has done quite limited work and just now is marking time. It has done enough, however, to show that the size of the technical problems is very great, and that we still have far to go in exploration of physical fact and feasible engineering procedure in shore protection. This added investigation, naturally, is not likely to be undertaken unless there is genuine and widespread public interest to justify it.

The reasons for public apathy toward shore protection are not quite clear. One element doubtless is the fact that most people pass their lives far from the shore front and have only the momentary interest of visitors. Another, perhaps, is that public authorities in most regions have done little to make the shore a true public asset, and that the hazards which beset the private shore owner do not greatly stir the public imagination. It is interesting to note that the work of the Long Island State Park Commission has done more in two or three years to draw public attention throughout New York State to the importance of shore values than anything in the past history of the region.

A beginning is being made here and there to transform the country's coastal frontier into a public asset. By the time this movement becomes really extensive an active public interest in preserving shore values will assuredly develop. There is reason to look forward hopefully to such a condition, for, as in the case of city dock frontage, systematic or well-managed administration of such resources is hardly possible so long as ownership is dispersed among many private interests.

There is substantial reason for placing the community's interest in the shore above private interest. Shore conditions at one point intimately affect neighboring regions, as is well recognized, and experience daily gives further proof of the fact. Stabilization and development of any zone for best use can therefore be carried out only by aid of a broad-gage plan of joint administration. And coupled with the problems of administration and control are formidable technical questions still at issue.

Only a beginning has been made in solving the problems of shore actions and shore protection. It can hardly be said that there is as yet a science of shore protection, certainly it is not beyond the stage of earliest infancy. One of the greatest fields of engineering development still waiting to be explored lies in this subject, and the work of a great many men will be required for its effective exploration.

Coastal actions are protean in nature; conditions vary from point to point and from moment to moment, and a remarkably broad range of observation must be at hand before any useful generalizations can be drawn. Observation, experiment and full-scale trial all have their place in the work that will be necessary to build a well-developed art of shore protection. It is a hopeful sign, therefore, that the interest of engineers in the subject is growing. For this interest will in time assure that systematic and extended cooperation in study from which an efficient art may be expected to develop.

## Steel Breakwaters

DEVELOPMENT of the steel breakwater is a distinctive contribution of Great Lakes harbor engineering to marine construction. Beginning in 1931 at Waukegan, Ill., with a stone-filled structure consisting of parallel vertical walls of sheetpiling partitioned by sheetpile cross-walls, and followed a year later, also at Waukegan, by similar construction with the parallel walls having a batter of 1:6, practice has turned somewhat firmly to sheetpile cellular structures with arched outer walls and partitions. The largest of these newer structures, described in this issue, is at Calumet Harbor; later a similar structure was undertaken at Fairport Harbor on Lake Erie. With the steel-plate cylinder breakwater at Indiana Harbor, where the handling of 26x42-ft. cylinders weighing 23 tons proved discouraging to this method of construction, the structures named make the full count of sizable Great Lakes steel breakwaters.

The importance of this brief history of development is that it indicates remarkably fast progress of a radically new type of structure toward established design. At present the design of steel sheetpile breakwaters is a somewhat undetermined combination of rational computation and plain empiricism, but perhaps not greatly more so than is all breakwater design. Again, there is much more to be discovered concerning the durability of steel in water, particularly in the zone that is alternately wave-washed and dry. Finally, the security of the pile interlock when subjected to ice impact in storms is a matter in which further experience will be instructive. Experience with lighthouse piers in the open lake waters has shown that a strong armoring, as is also illustrated in this issue, is required to resist damage by ice hurled against the pier by storm waves. It is true that a safeguard against unlocking exists in the fact that with the present arched shell walls the interlock is under direct tension; nevertheless, ice action would seem to be the main hazard of the sheetpile cellular breakwater.

Casting up the total of present experience with steel breakwaters, it is found that they cost a little less than other types, have probably greater bulwark strength, have distinct construction weakness until the cells are filled, present a good appearance and leave uncertain several questions of interlock integrity and resistance to corrosion. Altogether the count indicates real worth.

# CURRENT NEWS

## Court Rules Federal Condemnation of Land for Housing Unlawful

A DECISION handed down by the sixth district United States Circuit Court of Appeals at Cincinnati on July 15 denies the right of the government to condemn land for low-cost housing and slum clearance. The decision upholds the ruling of U. S. District Court Judge Charles I. Dawson of Louisville, Ky., that slum elimination is not "public use." Judge Dawson's ruling, on Jan. 4, halted work on the $1,618,000 PWA housing project in Louisville.

The majority opinion of the circuit court, to which the case was appealed, was written by Judges Moorman and Hicks. The opinion held that although there is no constitutional authority for the exercise of eminent domain by the government the principle is well established but only when exercised in taking land for public use. The decision states that "The contention of the government is that the property here sought to be condemned is to be devoted to a public use because, first, the construction of the project will relieve unemployment during the period of construction, and secondly, the leasing or selling of the new buildings at reasonable prices will give to persons of low income an opportunity to improve their living conditions. We do not think the first of these purposes, if made effective, could be said to constitute the use to which the property is to be put."

It was further held by the majority report that the act, under which the housing program is being carried out, contains no definition or standard of what should constitute slum clearance or low-cost housing projects. It was also pointed out that no limitation or requirement was imposed upon the administrator with reference to the spending of money and the location of projects.

Judge Florence Allen, in her minority opinion, upheld the housing program by maintaining that the power of eminent domain could be exercised wherever necessary for carrying into execution the power of taxation and appropriation for the general welfare. Specific approval of slum-clearance was given in Judge Allen's dissenting opinion.

### The next step

Administrator Ickes pointed out that there remain two general methods of procedure on slum clearance projects that can be utilized without overstepping the ruling of the court. One is cooperation with municipal or state authorities, to permit local condemnation proceedings; the other, direct purchase of land by the federal government by negotiation without condemnation. The court ruling did not challenge the right of the government to install low-rent housing, but confined itself to the method of land acquisition.

Under the direction of A. R. Clas, Director of the Housing Division, plans have been laid for continuing the PWA slum clearance program within the court ruling. It will be possible to install, on vacant land, housing projects which will be available to families now living in slum dwellings, thus taking families out of substandard housing without direct demolition of slum areas. In many cases cities have already agreed to demolish under their police power a number of substandard dwellings equal to the number of new housing units provided.

Secretary Ickes expects that the U. S. Supreme Court eventually will pass on the federal government's right to condemn land for low-rent housing, but he has not yet decided whether to appeal from the adverse decision. He asserts that the decision will make it difficult to carry forward housing projects in some communities, but that it will not seriously retard the program as a whole. "There is no reason to suppose," he said, "that another court in another jurisdiction will not arrive at a different conclusion."

## Yangtse River Flood in China Greatest in Recent Years

Reaching even greater proportions than the flood of 1931, the Yangtse River, in China, broke through dikes and levees and flooded about 50,000 square miles of land. Five miles of the Kiukiang-Nanchang Railway were washed out.

The collapse of a mud dike near Hankow wrecked hundreds of homes. The dike had been built in 1934 without the direction of a trained engineer. Several million inhabitants have been rendered homeless, and incalculable property damage has been caused by the flood, which, according to the last reports, threatens to become worse in the lower river valley.

## Contract Modification Clause Ruled Out by McCarl

*(Washington Correspondence)*

Comptroller-General McCarl has ruled out the so-called "blank check" clause in government contracts which was the administration's effort to continue NRA code requirements in force in so far as government purchasing is involved. The Treasury's Procurement Division wanted a blanket promise from bidders to abide by subsequent wage, hour and child-labor limitations proposed in the pending Walsh bill, presumably with no changes in contract prices. Mr. McCarl saw as a result of the clause unnecessarily high prices or subsequent demands for price readjustment. "Contracts obligating appropriations," he said, "must be definite and certain." He ruled that a clause calling for modification of the contracts to conform to later legislation may properly be regarded only as a request to bidders, not as a command. A low bid omitting this stipulation is still a valid bid.

## Ridgway Reports on Transit For City of San Francisco

Robert Ridgway, former chief engineer of the Board of Transportation of New York City, on July 12, submitted a report on the rapid transit problem of San Francisco to the Public Utilities Commission. Mr. Ridgway was assisted by Alfred Brahdy, designing engineer of the Board of Transportation, New York. His report was a review of a rapid transit plan tentatively formulated for E. G. Cahill, utilities manager, by a committee headed by Paul J. Ost.

Mr. Ridgway's recommendations follow the Cahill plan closely, but he proposes to modify or eliminate certain elements of that plan to reduce its estimated total cost of $56,200,000 by $3,500,000. He recommends the construction of eight and one-half miles of subway through which the present surface cars would be operated with slight modifications until such time as they are replaced by new equipment.

The principal elements of the plan are: (1) A two-track subway under Market St. to Church St. where surface connections are made with the existing lines through Twin Peak and Sunset tunnels, a distance of 2½ miles; (2) a branch from the Market St. subway at Van Ness Ave. under Mission St. and over the old Southern Pacific right-of-way and the Bernal cut to Monterey Blvd., two miles in subway and one and one-half miles on the surface; and (3) two and one-half miles of subway under Montgomery St., Market St. and Geary St. The easterly end of the Market St. subway line is to be turned south under Fremont St. to a transfer point for passengers from the transit lines on the Trans-Bay Bridge.

The Market St. subway would be designed so that it could be enlarged to a four-track line when required. In the initial step the tracks of the Municipal Railway would be transferred to the subway leaving the Market Street Company's tracks on the surface. Unification of the latter company's lines and those of the California Cable Co. with the Municipal Railway was urged.

## New Orleans Sewerage Project Held Up by Injunction

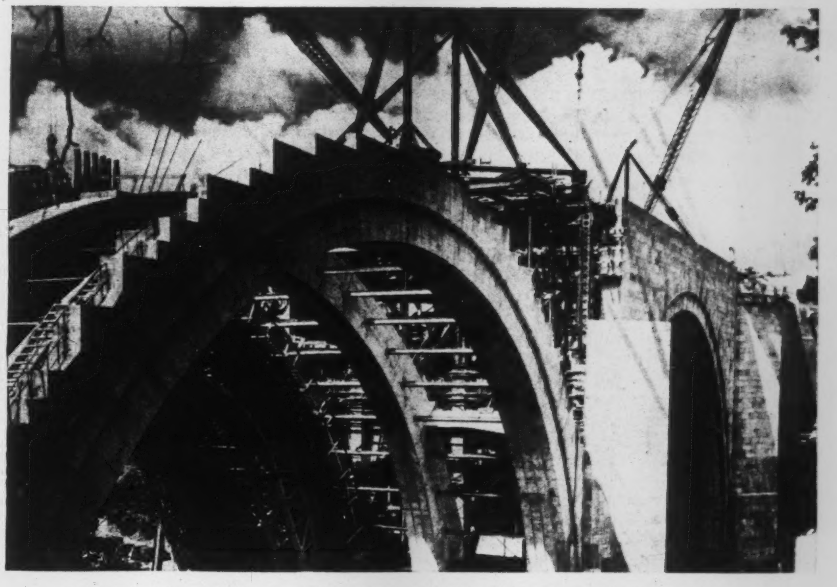
An injunction recently granted by the New Orleans civil district court, at the instance of the attorney-general's office, enjoins New Orleans banks from honoring drafts or checks of the sewerage and water board. Because the board is unable to make use of any of the PWA funds it has banked, contractors have abandoned work on the projects already started. Installments which were due on May 15 and June 15 are held up by the order.

Among some of the larger projects which were abandoned was the covering of the open drainage canal from Nashville Ave. to Leonidas St. Work was abandoned on June 29 by R. P. Farnsworth & Co., contractors, when the sewerage and water board failed to meet the installment amounting to about $30,000 which was due on the contract. The same company also abandoned work on a new drainage canal on Almonaster Ave. An installment of about $20,000 was due on this project. Work of installing a concrete pipe line on Baudin St. was abandoned by the contracting firm of Flynn & Co. when an installment of about $26,000 was withheld. Although contracts had been let for two large sewer projects, the contractors will not start work while the injunction is still in force.

The constitutionality of laws recently passed in a special session of the legislature calling for the reorganization of the sewerage and water board was attacked in the court. The ensuing litigation resulted in the issuance of the injunction.

## Puzzolan-Blended Cement May Be Tried by TVA

Plans are being developed to make a trial of puzzolan-blended portland cement in one section of the TVA's Norris Dam, to determine by direct comparison with a check section of unblended portland whether the two materials behave differently as regards shrinkage and other practical factors of service. The plan is to substitute for 25 per cent of the Norris Dam cement (a moderate-heat cement, intermediate between Boulder Dam and standard portland) an equal amount of slag cement or puzzolan cement. Details of where and how the trial can be made with least interference to construction remain to be worked out.



CALVERT ST. BRIDGE ACROSS ROCK CREEK VALLEY, WASHINGTON, D. C.

A new three span concrete-arch bridge is being built across Rock Creek at Calvert St., Washington, D. C., to replace an old steel viaduct which was moved 80 ft. to provide a detour for traffic while the new structure is being completed (*ENR*, Feb. 21, 1935, p. 282).

The new bridge when completed will have a length of 751 ft. and a maximum height of 125 ft. from ground to the deck. Exposed areas of the abutments and piers as well as the outside of the arches are faced with limestone which is also carried up to deck level as curtain walls over the arches. In order to give the appearance of a solid concrete arch from below, precast slabs were placed between the arch ribs.

The bridge, which was designed by Modjeski, Masters & Case, is being built under the direction of H. C. Whitehurst, director of highways, and C. R. Whyte, engineer of bridges, District of Columbia. Construction work is being done by the John W. Cowper Co., Buffalo.

## President Approves Relief-Works Rules for PWA Projects, Roads and Crossings

REGULATIONS for the organization and prosecution of relief works have been approved and signed by President Roosevelt. Four documents were issued; one applicable to grants for PWA works, one applicable to grants and loans, a third for highways and a fourth for grade crossings. In general the organization and procedure are alike for both types of PWA projects and alike for roads and crossings and all are similar in broad labor provisions. Only major requirements are noted here.

### PWA projects

The new procedure governing PWA projects provides for examination of projects in the states, a simpler form of contract consisting of an offer by the government and acceptance by the applicant, advanced payments of a portion of the grant which has been liberalized from 30 per cent of the cost of labor and materials to 45 per cent of the total cost of the project and predetermination of minimum wage rates based on prevailing local rates.

Important among the new rules and indicative of the increased responsibility of PWA state directors is the regulation covering wage rates which takes the place of the PWA three-zone minimum scale previously in effect. The primary responsibility of fixing wage rates on PWA projects under the new program rests with the borrower or recipient of a grant. Also, "if, in the opinion of the state director, work of any specific type has generally been done under union conditions in the political subdivision in which the project is to be constructed, the state director may disapprove any rate to be paid on such type of work, other than a rate predetermined under law or ordinance, if it is less than the present prevailing union rate."

Workers will be obtained through the United States Employment Service. Preference in employment shall be given to persons from the public relief rolls, and, except with the specific authorization of the WPA, at least 90 per cent of the persons employed on any project shall be taken from relief rolls. The contractor, however, may continue to bring supervisory and administrative employees and key men on the job, as at present, provided their number does not exceed 10 per cent of the total number employed. The contractor may dismiss any employee not qualified to perform the work for which he has been hired.

The rules prescribe that except in emergencies the maximum hours of manual work on PWA projects shall be 8 hours per day and 130 hours per month. In some cases a 40-hour week will be permitted. Wherever practicable, double shifts of labor will be employed in order to speed construction and spread employment.

Unless otherwise provided by law, claims or disputes pertaining to the classification of labor under the construction contract shall be determined by the applicant, subject to final review by the PWA state director whose decision shall be binding.

All employees shall be paid in full at least once each week. All pay rolls shall be sworn to in accordance with regulations issued jointly by the Secretary of the Interior and the Secretary of the Treasury pursuant to the so-called "kickback" statute.

The new regulations provide that any time after acceptance by the applicant of an offer by the government to aid in financing a PWA project, the applicant may request an advance payment of the grant of not exceeding 15 per cent of the previously approved estimated cost of the project. This advance grant may be used for paying architectural, engineering, planning and legal fees, costs of surveys, borings and other preliminary investigations, costs of preparation of plans, specifications and other forms of proposed contract documents, the costs of advertisements for bids for contracts and the printing of bonds, but not in payment for the acquisition of lands, easements, or rights-of-way. The request for this advance grant must be accompanied by a signed certificate of purpose.

The rules provide for inspection of all work by the PWA Inspection Division and for reports on construction and progress and monthly reports to the Department of Labor covering the number of men at work, man-hours worked, the aggregate amount of payrolls and an itemized statement of expenditures for materials.

### Highways

The rules as promulgated by the Secretary of Agriculture through the Bureau of Public Roads, specify the methods by which projects are to be originated, checked, approved and put under way. All state programs must originate with the state highway departments. The state highway department submits its program to the district engineer of the Bureau of Public Roads, and with his approval, to the state administrator of the Works Progress Administration and the state director of the National Emergency Council. Having received their concurrence, the program is then submitted by the district engineer to the Bureau of Public Roads in Washington, where final approval and allotment of funds will follow.

Under a special provision of the apportionment of the $200,000,000 for public roads, the figure allowable per man-year of employment is $1,400. However, a state may propose an alternate plan of highway work and have it accepted, provided it meets the requirements in reducing the relief load.

Not less than 25 per cent of the state's apportionment must be spent on construction of rural secondary or feeder roads which are not on either the state or federal-aid systems, and not less than 25 per cent of the apportionment shall be applied to projects within municipalities or metropolitan areas. A state is required to spend not less than 1 per cent of its apportionment for improvement of the roadside. An amount not to exceed 1½ per cent of the fund may be used to finance a planning program.

Three grades of labor are established, unskilled, intermediate and skilled. The minimum wage rates established shall be reasonably comparable to the hourly rates paid by the state highway department with its own funds for work of a similar character. Preference in employment of labor shall be given (except in executive, administrative, supervisory and highly skilled positions) to persons from the public relief rolls, and at least 90 per cent of all persons working on a contract shall be taken from public relief rolls. Hours of work are substantially the same as have been established for PWA projects.

### Grade crossings

As in the case of highway construction, the grade crossing projects must originate with the state highway department. This department submits the program in parcels of not less than 25 per cent of the whole state program to the district engineer of the Bureau of Public Roads. If satisfactory to him it is then laid before the state administrator of the Works Progress Administration and the state director of the National Emergency Council for their concurrence before the district engineer submits it to the Bureau at Washington. In the absence of a specific waiver by the Secretary of Agriculture, not less than 25 per cent of each state's apportionment is required to be expended for elimination of hazards at crossings on secondary or feeder roads, including city streets which are not included in the federal-aid highway system or its municipal extensions.

Four types of projects are included in the program: Protection of grade crossings, separation of grades at crossings, reconstruction of existing railroad grade crossing structures and the relocation of highways to eliminate grade crossings.

No projects will be financed with federal funds on new highways which are not relocations of existing highways. Protection of crossings may be either by lights or barriers of a nature acceptable to the state highway department and the railroad affected. To secure a reasonably equitable distribution of the benefits to the various railroads in any state, crossing projects will be apportioned according to the proportionate mileage of the railroad to the total mileage of all railroads within the State. In some cases relocation of highways will accomplish the ends of safety, and if the cost of relocation does not exceed the cost of structures for the separation of the grades, this method will be allowed.

Alternate rules are prescribed for determining the acceptability of grade crossing projects proposed by the states. Under the first rule the total cost to the federal government, including labor, material and other items, may not exceed $1,400 per man-year of direct employment. Under the second and alternate rule, 40 per cent of the total cost, including costs of private property, must go to persons directly employed on the project.

## TVA Starts Work on High Line To Pickwick Landing Dam

The Tennessee Valley Authority has started work on the construction of a 300,000 high tension power line extending from the Wilson Dam to the site of the Pickwick Landing Dam. This line, upon completion, will be about 45 miles long and will cross the Tennessee River about two miles down stream from the Wilson Dam and then extend along the north bank of the river to the site of the proposed power house at the Pickwick Landing Dam. Power will be supplied to the construction areas on both sides of the river at the Pickwick site. For about five miles from the Wilson Dam, the line will be supported by steel poles; for the rest of the line wood-pole H-type structures will be used.

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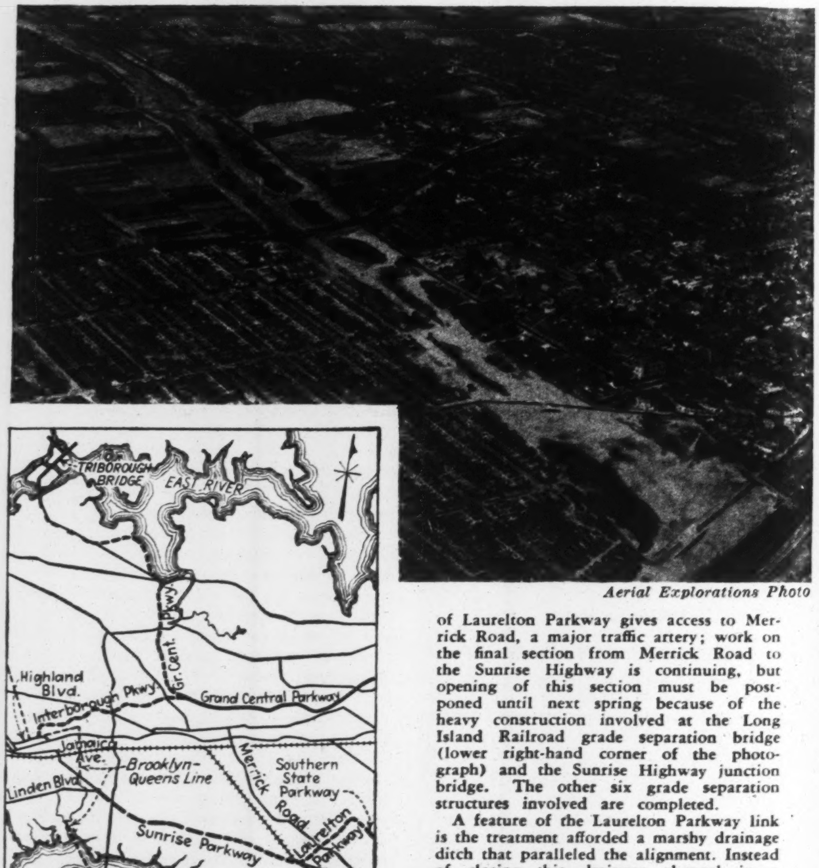
### NEW LONG ISLAND PARKWAY LINK OPENED TO TRAFFIC

**Completion of the first half of the 3-mile long Laurelton Parkway on July 2 brings the western terminus of the Southern State Parkway on Long Island that much nearer to Brooklyn, its eventual terminus. As shown on the accompanying map, the Laurelton Parkway links the Southern State Parkway with the existing Sunrise Highway, which is also in course of reconstruction to parkway specifications, to provide two three-lane roadways separated by a 10-ft. grass strip. The completed section**



*Aerial Explorations Photo*

**of Laurelton Parkway gives access to Merrick Road, a major traffic artery; work on the final section from Merrick Road to the Sunrise Highway is continuing, but opening of this section must be postponed until next spring because of the heavy construction involved at the Long Island Railroad grade separation bridge (lower right-hand corner of the photograph) and the Sunrise Highway junction bridge. The other six grade separation structures involved are completed.**

**A feature of the Laurelton Parkway link is the treatment afforded a marshy drainage ditch that paralleled the alignment. Instead of closing this drainage channel in a buried conduit, a series of pools were formed by small overflow dams built in connection with the access bridges that span the drainage course.**

**With work on the Interborough Parkway to the north heading toward a completion date in 1936, direct access from Brooklyn to both the Northern State (Grand Central) and Southern State Parkways will soon be provided.**

## Reclamation Bureau Gets $100,000,000 Allotment

The Advisory Committee on Allotments has formally adopted a resolution calling for about $100,000,000 to be made available to the Bureau of Reclamation stating that "these funds shall be transferred after conference with the Secretary of Agriculture and appropriate decisions have been made relative to a proper balance between land which is brought into cultivation by these projects and an equivalent amount which shall be taken out of cultivation."

## Treasury Order Halts Work On 27 Post Offices

A default order issued on July 1 by the Treasury Department against the Murch Brothers Construction Co. of St. Louis and Washington, D. C., terminated the right of the company to proceed on 27 post office projects. The order was issued because of lack of progress by the company on its construction jobs, a situation generally said to have resulted from the fact that the company had been the successful bidder on more jobs than it could financially handle.

## SOCIETY CALENDAR

PUBLIC WORKS ENGINEERING CONFERENCE, joint meeting of the American Society of Municipal Engineers and International Association of Public Works Officials, Cincinnati, Ohio, October 14-16.

INSTITUTE OF TRAFFIC ENGINEERS, sixth annual meeting at Louisville, Ky., October 15-16.

AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS, annual convention, Miami, Fla., December 9-12.

AMERICAN WATER WORKS ASSOCIATION, Central States Division, Pittsburgh, Pa., August 22-24.

NEW ENGLAND WATER WORKS ASSOCIATION, fall convention, Providence, R. I., Sept. 17-20.

# Shore Protection Problems Discussed At Ocean City, N. J., Meeting

Recognition of cooperation in financing and administering shore protection, practical experience in shore problems and new field studies were leading subjects of discussion at the Ocean City, N. J., meeting of the American Shore and Beach Preservation Association, July 11 to 13, which was attended by about 75 members. Gen. Edward M. Markham, chief of army engineers, and a number of representatives of the U. S. Beach Erosion Board were present.

Summarizing the experience of New Jersey, which has long been among the most active sectors of the United States coastal region in shore-protection efforts, Mayor J. G. Champion of Ocean City said that little effective work can be done by individual shoreowners or small groups, and that extensive cooperation is essential. Former Attorney-General Stevens of New Jersey emphasized the same point by detailed reference to experience along the north Jersey coast, where beaches exist only through the effect of groin construction. Originally the individual property owner undertook to protect his own shore frontage, but this was so ineffective that about 20 years ago municipalities were authorized to build protective works in front of private property and assess benefits. Later still, state aid was extended to local communities through the Board of Commerce and Navigation. Even this extent of cooperation has not proved sufficient, recent storms having destroyed great amounts of property.

### Legal problems

Legal questions in shore matters were discussed more specifically by J. Raymond Tiffany, deputy attorney-general of the state. The common-law right of the individual owner to construct bulkheads or groins to protect his property even where injury to a neighbor may result is well established by decisions. An oil company that built a wall into tidal waters on its land adjoining a costly yacht basin development, causing sand to be deposited in the basin, was held not liable. Negligently constructed walls or groins might give rise to claim, however. The individual owner is free to take material from his land above high water; but dunes or shingle masses that form a protection against the sea may not be removed if the adjoining owner is injured.

Gen. Markham, speaking on the possibility of applying some of the federal relief works fund to shore protection, said that, in view of the long-time continuing nature of shore-protective work, "reaching for emergency money is no good." He also criticized the states for inaction; the Beach Erosion Board, set up by the federal government for cooperative work with states, is not doing much business for lack of customers, he said; the states, who should be customers for its work, are not buying. He stated also that the association has done little to work out a basis for cooperation.

A specific proposal for bringing shore protection into the federal relief works program was made by T. A. Scott, of Merritt-Chapman & Scott Corp., New York. It was directed toward protection of Ambrose Channel, the entrance to New York harbor, from sand inflow from the south shore of Long Island, which is subject to rapid continuous westward drift of sand derived from erosion of the Montauk Point area. Ambrose Channel was filling up in 1933, when the Rockaway Point jetty (*ENR*, May 4, 1933, p. 547) was being built to hold back the sand drift. In two years the jetty has impounded some 6 million yards of sand, but now sand is beginning to pass around its end. If a definite protective project were agreed to by Nassau and Suffolk counties on Long Island, and by the north shore authorities of New Jersey, it would be possible to put a large number of the unemployed of the metropolitan area to work, provided an ample supply of stone at low cost could be assured.

### Technical developments

A study program to provide basis for the design of a jetty system to maintain navigation through Absecon Inlet, N. J., was outlined by Col. John C. Lee and Capt. J. Kohloss, Philadelphia. The facts that will be recorded continuously in the Absecon study relate to beach profiles, tide and wave heights, water currents, wind velocity and direction, cross-sections of the gorge of the inlet, bar samples, etc. Recording tide-gage observations are to be carried back into the bay in order to give data on water slopes.

Col. E. J. Dent, U. S. District Engineer at Baltimore, described the formation of the intermittent, shifting inlet through the barrier beach at Ocean City, Md. Such inlets are not formed by sea attack, he believes, but by outflow from the lagoon or bay in the rear. To stabilize conditions and to provide for fishing-vessel navi-

*(Continued on p. 102)*

## Washington Highlights

### *By Paul Wooton*

Chances are slim for Mississippi river flood control legislation at this session of Congress. The House Committee is badly split on the proposal of its chairman that the non-controversial phases of the Markham plan be approved. Gen. Markham will not object if Congress enacts only a part of his recommendations at this time, particularly since that would enable him to finish the work in the Atchafalaya basin.

Support for the exaggerations of the columnists as to lobbying by officials of the executive branch is furnished by the Corcoran-Brewster incident. It will be hard to convince the public now that attempts to bring improper pressure on legislators are rare.

No grants will be made by the Rural Electrification Administration, Morris L. Cooke, its administrator, announces. Advances to privately-owned utilities, to publicly-owned plants, and to farm cooperatives, he states, will be on a loan basis. Interest will be 3 per cent, with twenty years for repayment. Erection of lines, Mr. Cooke says, will be by contractors, utility companies, public agencies, farmers and by the federal government itself.

At least ninety days must pass after the approval of the banking act before the Reserve system can begin operation under a new charter. Selection of new officials, formulation of new regulations and reorganization of the open market committee take time.

*Washington, July 16, 1935.*

## Work on Boulder Dam Halted by Strike

All work on the Boulder Dam project in Nevada was halted on July 13 when truck drivers joined the striking carpenters and steel workers in their demands for a 7½-hr. day. A recent order by the Six Companies, Inc., contractors on the job, required that the men work 8 hr. a day with the ½-hr. lunch period on the men's time. Strikers demand a 7½-hr. day with lunch on the company's time. According to the Central Labor Council, to which the dispute was submitted, the trouble has been smouldering for more than a year because the contractors would not grant the workers' demands for wage increases.

E. H. Fitzgerald of the Los Angeles office, U. S. Department of Labor Conciliation Service, has been ordered by Secretary Perkins to act as mediator of the Boulder Dam strike.

*(Continued from page 101)*

gation, a pair of jetties is under construction. Drift is predominantly to the south in this reach, for which reason the north jetty was designed to be a sand trap; in a year since completion of the body of the embankment it has impounded 200,000 cu.yd., kept sand out of the inlet, and extended the beach 200 ft. The jetty was not built on the sand-tight principle followed by New Jersey. The core (up to low water) was built with small stone included, using as much of the quarry output as possible, and a blanket of 6-ton stone was laid over it. On top of the embankment, which extends up to about mean high water, a concrete wall 5 ft. high and 4½ ft. wide at the base is being built. The length of the structure is about 1,500 ft., including several hundred feet of stone mattress at the outer end. The sand deposits on the windward side of the jetty are forming in depths as great as 20 to 24 ft., and as 10 ft. of sand has accumulated in 20 ft. depth, Col. Dent believes that the economic length of such a jetty is greater than had been thought.

On the question of sand-tight construction V. Gelineau, director, New Jersey Board of Commerce and Navigation, remarked that the jetty at Longport, south of Atlantic City, N. J., had been designed as not sand-tight, but in fact is tight because of the small material in the core.

In a later discussion forum, Col. Dent also summarized the groin recommendations of the Beach Erosion Board and warned against building too wide a beach. He characterized New Jersey as a natural laboratory in beach action, especially the steeper beaches of the north coast. Experimental laboratories, he said, are of doubtful value.

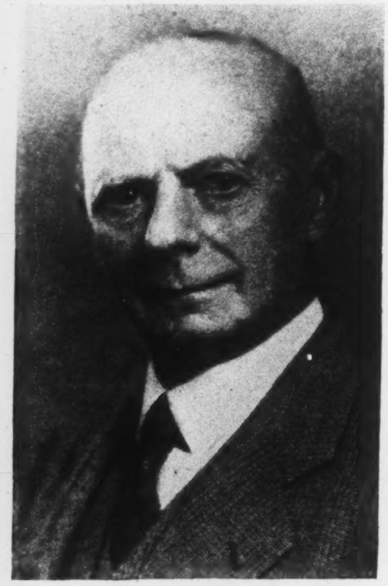
Certain of the difficulties inherent in shore operations, whether to deepen a channel or to form a beach, were pointed out by F. E. Schmitt, editor, *Engineering News-Record*. Such operations after changing naturally stabilized conditions seek to attain stability at a new point by reliance usually on natural forces alone. Since these forces are constantly varying, according to type of weather and other influences, the stability secured will not be absolute, he said, and a range of variation must be contemplated. Further, knowledge of the active forces is imperfect because observation of natural conditions can show only differential or residual effects, as in the case of gain or loss of a beach, where the net result is the difference between accretion and wastage in a large, continual movement of material. For these reasons it is not generally possible to compute or predetermine the effect of a proposed structure or other change, and laboratory experimentation should be a valuable guide in indicating the probable effects.

On the details of structures, Ralph Buvinger of Atlantic City argued the merits of permeable jetties on the lee side of inlets, and referred to the work of L. M. Haupt in this field. Progress in adaptation of steel sheetpiling to shore-protective structures led to some discussion. The chief improvements have been to deepen the arch webs for strength and to add copper for increased life, according to T. R. Smith, of the Carnegie Steel Co. Questions as to protection from erosion by sand and gravel led to the suggestion that face protection by creosoted timber is the best available.

## D. C. Henny, Hydraulic Engineer, Dies in Portland, Ore.

David Christian Henny, one of the country's ablest and best-known consulting hydraulic engineers and a leading figure in reclamation during its early days, died at his home in Portland, Ore., on July 15 after a brief illness. Mr. Henny was 74 years old. His death followed only six weeks after that of Mrs. Henny.

Coming to the United States in 1884 from his native country of Holland, where he was educated in the Polytechnic University of Delft, Mr. Henny engaged in general engineering work until 1892, when he became associated with the Excelsior Wooden Pipe Co., later the Redwood Manufacturers Co., of San Francisco, which he served as general manager for ten years. From 1905 to 1909 Mr. Henny was supervising engineer of the northwestern district of the U. S. Reclamation Service, resigning to begin his long and productive practice as a consulting engineer. He served the Bureau of Reclamation as adviser on practically all of its extensive dam building in the Northwest. In 1927 he was one of three members of a board appointed to report on the proper design and location of Owyhee Dam in Oregon, Deadwood Dam in Idaho and Gibson Dam in Montana. Mr. Henny was consulting engineer to the Los Angeles County Flood Control District at the time of his death and was also chairman of the Bonneville Dam Commission and a member of the consulting board for Fort Peck Dam.



David C. Henny

Mr. Henny served as vice-president of the American Society of Civil Engineers in 1932-33, and was chairman of the irrigation division for a number of years. He was distinguished by a deeply studious attitude toward every problem placed before him, by a keen power of observation and fearless originality of view. These qualities, coupled with high integrity and a frank, kindly nature, made him a leading figure in the engineering life of the West.

# *Brief News*

THE STATE CONTROL BOARD of Ohio, in its first meeting since temporarily withholding action on the request of the Muskingum watershed conservancy district for funds, authorized release of $350,000 for purchase of land. The $34,000,000 Muskingum watershed conservancy project is now employing 2,000 men. Although it will be about two years before the 14 dams will be finished, rapid progress is being made in the first stages of the undertaking. Work is under way on eleven of the fourteen dams.

THE MEXICAN GOVERNMENT has placed about $420,000 at the disposal of the ministry of communications and public works for flood prevention operations along the Rio Grande in the Matamoros region, Tamaulipas state. This work includes the construction of two irrigation canals to divert surplus waters of the river. Work is expected to start this summer.

# *Personals*

R. B. CHANDLER, civil engineer of Port Arthur, Ont., has been appointed manager of the Port Arthur Public Utilities Commission.

RHOTEN H. CLIFT has resigned his position as division engineer in the Tennessee highway department to become assistant WPA administrator in that state.

J. R. IAKISCH, with the U. S. Bureau of Reclamation, on the Pine View dam in Ogden canyon, Utah, has been advanced to the rank of construction engineer. Mr. Iakisch has been in charge of the Ogden canyon reservoir project for over two years.

LIEUT. PAUL W. THOMPSON, Corps of Engineers, U. S. Army, has been relieved from duty as assistant to the district engineer in Omaha, Neb., and assigned to the American Embassy in Berlin, Germany. He was ordered there for one year to study European hydraulic research methods under a scholarship awarded him by the American Society of Civil Engineers.

HAROLD C. FISKE, who was recently employed as division engineer by the Board of Transportation of New York City, has accepted a position as chairman of the Electric Power Board of Chattanooga, Tenn., recently created to acquire and operate a publicly owned electric power distribution system for the city of Chattanooga.

ROBERT B. BROOKS, JR., who has been an assistant engineer with the Mexican National Highway Authority, has resigned that position to become associated with his father, Robert B. Brooks, consulting engineer, St. Louis, Mo. While with the Mexican Highway Authority, Mr. Brooks served for a time as resident engineer on highway bridge pier and foundation construction.

ABUNDIUS BRULE, former city engineer of Central Falls, R. I., has been appointed engineer in the division of roads and bridges, state department of public works.

**JOHN E. GOODWIN**, Augusta, Me., has been appointed inspector of dams and reservoirs in that state.

**JOHN W. TAYLOR** has become resident engineer for the state highway department at Austintown, Ohio.

**RICHARD G. TYLER** has resigned his position as dean of the College of Engineering, University of Washington. Dean Tyler, who will be succeeded by **EDGAR A. LOEW**, will continue as head of the department of civil engineering.

## *Obituary*

**WILLIAM H. FULLER**, 96, died at his home in Palmer, Mass., on July 7. Mr. Fuller, who had long been retired from active practice, received his engineering education from Union College and later took graduate work at the University of Wisconsin. He spent a number of years in railroad location and construction in the Northwest.

**THOMAS MOORE KELLOGG**, member of the firm of Rankin & Kellogg, architects, Philadelphia, Pa., died at his home there on July 7 at the age of 73. Mr. Kellogg participated in the design of many buildings, among which were the Philadelphia Inquirer Building, and the Montgomery County Courthouse Annex at Norristown, Pa.

**JOHN NICHOLAS AMBLER**, 67, estimating engineer for the Virginia highway department, died recently at his home in Richmond. He was at one time city engineer and director of public works at Winston-Salem, N. C. He came to Richmond in 1918 as engineering instructor for an Army training school. Later he became a member of the staff of Virginia Mechanics Institute.

# CONSTRUCTION STATISTICS OF THE WEEK

HIGHER FEDERAL AWARDS have raised this week's total of engineering construction awards to $23,607,000. Of this total $10,552,000 represents state and municipal, $5,816,000 federal and $7,239,000 private construction. The awards for the corresponding week last year were $12,017,000 for state and municipal, $34,330,000 federal (Grand Coulee Dam and power plant contract included), and $9,929,000 private, making a total of $56,276,000.

Sewers at $3,383,000 are considerably higher than for several weeks, but all other classifications are about the same.

The larger contracts include a store and office building at San Francisco, Calif., for L. R. Lurie estimated to cost $500,000; housing development at Gates, N. Y., for Development Corp., Rochester, N. Y., $750,000; hydro electric power house at Columbus, Neb., for Loup River Public Power Dist., $613,000; steel work for Bronx Approach to Triborough Bridge, Contract 38, New York, N. Y., $1,094,000; joint outfall sewer, Schedule 1, 2 and 3 and White Point Ocean Outfall Sanitary Sewer for Los Angeles County Sanitation Districts, Los Angeles, Calif., $2,526,000. The U. S. Corps of Engineers at Zanesville, O., has awarded contract for Pleasant Hill Dam, Perryville, O., $919,000 and Bolivar Dam at Bolivar, O., $1,546,000; and at Kansas City, Mo., for dikes and revetment work in the Missouri River between Iowa and Missouri, $513,000; Iowa and Nebraska, $765,000; Missouri and Nebraska, $604,000.

New capital for the week includes state and municipal bond sales, $5,325,000; PWA grants for non-federal work, $5,111,000; corporate security issues, $855,000; and allotments for federal work totalling $33,835,000. The latter figure includes an arbitrary percentage (25%) of allotments for WPA (Works Progress Administration) work, part of which will be new construction or major additions. A review of corporate financing for the first half has brought out construction financing totalling $17,144,000 which has been added to the new capital total this week. This includes $11,226,000 for capital expenditures out of the $50,000,000 issue floated by National Steel Corp. in May and $5,000,000 for new plant construction out of the $25,000,000 issue floated by American Rolling Mill Co. in June. The distribution by months of the $17,144,000 addition through corporate security issues is: April, $668,000; May, $11,226,000; and June, $5,250,000.

### CONTRACTS

(Thousands of Dollars)

| | Weekly Average | | Week |
|---|---|---|---|
| | July 1934 | Prev.4 Weeks | July 18 1935 |
| Federal Government | $10,224 | $6,150 | $5,816 |
| State and municipal | 14,816 | 12,549 | 10,552 |
| Total public .... | $25,040 | $18,699 | $16,368 |
| Total private .... | 4,450 | 6,687 | 7,239 |
| Week's total .... | 29,490 | $25,386 | $23,607 |

Cumulative to date:
1934...$801,493,000 1935...$718,220,000

### NEW PRODUCTIVE CAPITAL

(Thousands of Dollars)

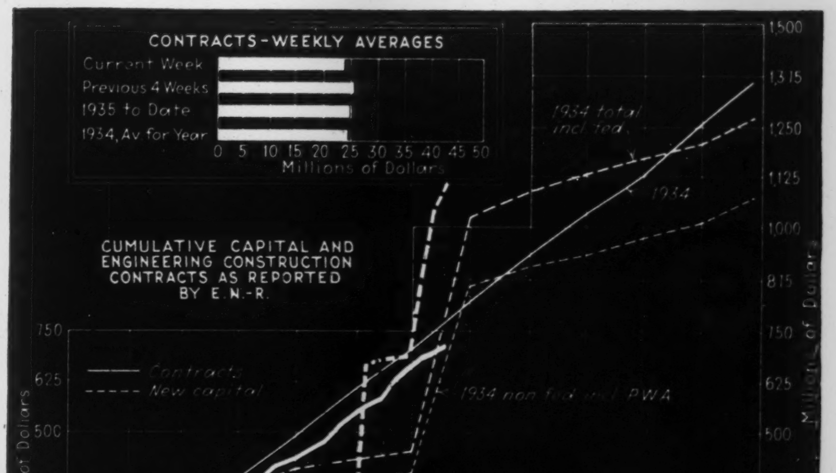
| 1935 | Week July 18 | Cumulative |
|---|---|---|
| State and municipal... | $5,325 | $208,137 |
| PWA allotments, S&M | 5,111 | 185,341 |
| RFC loans, S&M...... | 36,000 | 36,084 |
| Corporate issues ..... | 855 | 11,291 |
| PWA allotments, private ............... | ...... | 8,629 |
| Total, Non-Federal.. | $11,291 | $456,190 |
| PWA allotments, Federal Constr......... | 33,835 | 651,824 |
| Total new capital... | $45,126 | $1,108,014 |

Cumulative to date:
1934..$982,925,000 1935..$1,108,014,000

Note: These figures include private bonds, and stocks sold for productive purposes; state and municipal bonds for construction; PWA loans and grants to states and municipalities, including the special highway funds; PWA private loans, and allotments for Federal construction. An arbitrary percentage (25%) of the WPA allotments is included to allow for the capital additions through the Works Progress Administration division of the new program.

### INDEX NUMBER

| E.N.R.-Cost | 1913 = 100 | 1926 = 100 | E.N.R.-Volume | 1913 = 100 | 1926 = 100 |
|---|---|---|---|---|---|
| **July, 1935...** | **195.18** | **93.82** | **June, 1935...** | **122** | **54** |
| June, 1935... | 194.78 | 93.63 | May, 1935... | 109 | 48 |
| July, 1934... | 199.65 | 95.97 | June, 1934... | 119 | 50 |
| 1934 (Av.)... | 198.10 | 95.23 | 1934 (Av.)... | 114 | 50 |
| 1933 (Av.)... | 170.18 | 81.80 | 1933 (Av.)... | 102 | 47 |
| 1932 (Av.)... | 156.97 | 75.45 | 1932 (Av.)... | 127 | 56 |



# *Construction Equipment and Materials*

## Road Equipment Show Sponsored in Buffalo

A road equipment show is being sponsored in Buffalo, N. Y., July 16-19, by Dow & Co., Inc., dealers in highway construction equipment.

Products of the following organizations are being displayed at this show which is the first of a series of annual shows being planned by the Dow company:

International Harvester Company
Thew Shovel Company
C. H. & E. Manufacturing Company
Gravely Motor Plow Company
Wellman Engineering Company
Brookville Locomotive Company
Mohawk Asphalt Heater Company
Frank G. Hough Company
Detroit Harvester Company
Western Cataphote Company
Williamsport Wire Rope Company
J. D. Adams Company
T. L. Smith Company
Barco Manufacturing Company
Chicago Pneumatic Tool Company
The Burch Corporation
Euclid Road Machinery Company
Metal Forms Corporation
Ready-Power Company
Illinois Wire & Mfg. Company
Empire State Culvert Corporation
St. Louis Steel Products Company

Factory representatives of many of the exhibitors are in attendance with demonstrators.

Moving pictures and other features are included in the show.

## New Elevating Grader Equipped With Hydraulic Controls

A new 3-wheel elevating grader with hydraulic operating control has just been developed by The Austin-Western Road Machinery Co., Aurora, Ill.

By supporting the welded steel frame on three wheels, the weight of the fully loaded elevator is counterbalanced so that the 32-in. disk plow can be forced down and worked to maximum capacity. The right rear wheel is laterally adjustable so as to increase its tread.

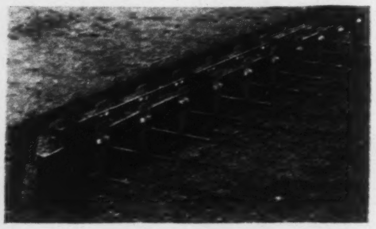
The stiff leg plow is equipped with an extra strong beam and standard. The beam is arched directly ahead of the plow to eliminate choking when handling sod or large clods. The plow can be set to float without affecting the lifting operation, and deep cuts or wide shallow cuts are made by varying the position of the furrow wheel.

The elevator, driven at high speed by a special universal shaft, gets the material away from the plow as fast as it is dug. A troughed belt with antifriction rolls does away with the necessity for full length side boards on the elevator. Adjustment to the elevator as well as to the plow and furrow wheel are made by hydraulic power control.

Austin-Western Elevating Grader.

## Expansion Joint Spotter Assures Perfect Alignment

A combination dowel, tie-bar and expansion joint spotter has been developed by the Flexible Road Joint Machine Co., Warren, Ohio. The frame holds the dowel bars in parallel alignment and keeps the expansion joint vertical while materials are being set. By the movement of a series of cams the frame can be released and then carried to another joint. The manufacturer believes that by keeping the dowel bars parallel the frame reduces the cracking of the concrete behind the dowel bars. The joint spotter is of light weight and can easily be handled by workmen on the job.



Expansion Joint Spotter for Aligning Dowel Bars.

## *New Publications*

NON-SKID ROLLED STEEL FLOOR PLATES, Central Iron & Steel Co., Harrisburg, Pa. 4x9, 8 pages.

HELTZEL STEEL FORMS FOR MODERN ROAD CONSTRUCTION, Heltzel Steel Form & Iron Co., Warren, Ohio. 8½x11, 8 pages.

BLAW-KNOX ROAD FINISHERS, Blaw-Knox Co., Pittsburgh, Pa. 8½x11, 24 pages. Bulletin No. 1507.

AUSTIN-WESTERN ELEVATING GRADER, Austin-Western Road Machinery Co., Aurora, Ill. 8½x11, 12 pages.

MUNLOCK BUILDING TILE INSURES, Munlock Engineering Co., Washington, D. C. 9x11½, 12 pages.

CONVECTOFIN, Commodore Heaters Corp., New York City. 8½x11, 12 pages. Bulletin 8.

GENERAL CATALOG, Manhattan Rubber Mfg. Div. of Raybestos-Manhattan, Inc., Passaic, N. J. 8½x11, 56 pages.

HOW TO HANDLE IT, Chain Belt Co., Milwaukee, Wis. 8½x11, 30 pages. Conveyor equipment.

SNYDER'S BLACK ROSENDALE CEMENT, The Wait Associates, Inc., New York City. 8½x11, 4 pages.

## *Business Notes*

WILLIAM GOODMAN, formerly consulting engineer of Chicago, has joined the engineering staff of The Trane Co., air conditioning equipment manufacturers, La Crosse, Wis.

REPUBLIC STEEL CORP., Youngstown, Ohio, has announced the removal of the Buffalo, N. Y., district sales office from 475 Abbott Road to 1020 Liberty Bank Building. Thos. B. Davies continues in charge of the office.

BUCYRUS-ERIE CO., South Milwaukee, Wis., announces the appointment of the R. S. Armstrong & Bro. Co., Atlanta, Ga., as distributor for the sale of power shovels, draglines, cranes, and skimmer scoops in Georgia.

THE BURNS & MCDONNELL ENGINEERING CO., announces that they have established a permanent office in Albany, N. Y. This eastern office will be in charge of Hartwell J. Rosson, who was formerly in charge of the Cincinnati office.

THE LOUIS ALLIS CO., manufacturer of electric motors has appointed the Harris-Green Company, Pittsburgh, Pa., as their engineering-sales representatives in the western Pennsylvania territory.

LINK-BELT CO., Chicago, Ill., has announced the appointment of Construction Equipment Co., 2274 Main Street, Hartford, Conn., as authorized distributors of Link-Belt crawler-mounted shovels, cranes, draglines, and track-type locomotive cranes.

## *New Equipment in Brief*

*Drafting Aids.* Isometric lined paper with axes spaced 1 in. and subdivided to ¼ in., and perspective lined paper with a 7-in. prism graduated to ¼ in. are now being marketed by the Wade Instrument Co., Cleveland, Ohio. A new beam compass with micrometric adjustments and an irregular curve made of a spring steel core covered with beveled rubber are also among the Wade company's new instruments. The curve has a metal guide and it clamps in any desired position.

*Test-Light and Fuse Puller.* Ideal Commutator Dresser Co., Sycamore, Ill., now has available a combination Test-Lite and Fuse Puller with 18 in. flexible leads. This new tool will test circuits of from 110 to 550 volts and will test, remove or replace fuses from 30 to 100 amp. capacity. Also, it can be used for handling live electrical parts, adjusting loose cut-out clips and fishing for loose articles surrounded by live wires. The tool resembles a pair of pliers and is made from molded reinforced bakelite, a non-conductor; being only 7 in. long it can be carried in the pocket. The test pins are mounted in the handle ends, and are adjusted for various spans by opening and closing the handles. The test light is enclosed in one of the handles to safeguard it from damage.